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McNARY DAM – DESIGN FROM TECHNICAL CONSIDERATIONS

by H. L. Drake and G. C. Richardson,
Associate Members, ASCE, and
H. M. Rigler and R. A. Schuknecht

POWER DIVISION

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THIS PAPER

McNARY DAM—DESIGN FROM TECHNICAL CONSIDERATIONS

H. L. Drake[1] and G. C. Richardson,[2] Associate Members, ASCE, and
H. M. Rigler,[3] and R. A. Schuknecht[4]

SYNOPSIS

The design of McNary Dam, as of all major projects, is the result of the
coordinated effort, not only of many individuals, but of several different groups
of individuals in places as far removed as England, where two main genera-
tors and six main transformers are being constructed, and Portland, Oregon,
where design was initiated. A series of papers, of which this is the second,
is being presented on the McNary project. The first[5] covered the general
features, and subsequent papers will be concerned with construction problems.
The present paper discusses some of the more unusual and interesting design
problems which faced the Corps of Engineers, first in the Portland District
and later in the Walla Walla, Washington, District after it was formed in
November 1948. The design of a spillway with vertical lift split gates, the
highest single lift lock in the world, with all of its attendant problems, a
power plant rated at 980,000 KW, and the most modern of fish passing facili-
ties were some of the major tasks confronting the design staff. The discus-
sion is presented in four general sections, each from the point of view of its
author's specialty: foundations, hydraulics, structures and power facilities.

General Description

McNary Dam, on the Columbia River 292 miles from its mouth, is located
between the states of Oregon and Washington, at the foot of Umatilla Rapids,
which were formerly a considerable hazard to navigation, about 2-1/2 miles
above Umatilla, Oregon. It is the second of the Columbia River navigation
and power dams to be built by the Corps of Engineers, following Bonneville
Dam. The reservoir formed by the dam extends over 60 miles upstream,
about 25 miles above Pasco, Washington, and about 10 miles up the Snake
River, to the Ice Harbor Damsite. The dam consists primarily of a 14-unit
powerhouse of 980,000 KW nominal capacity, a 22-bay spillway having 50-ft.
x 50-ft. vertical lift gates, an 86 by 675-ft. navigation lock with a 92-ft. lift,
two fishladders, fishlocks, concrete non-overflow sections and rock fill abut-
ments, all of which make up a dam 7365 feet long. The major features of the
project are illustrated in Figure 1.

1. Chief, Foundations and Materials Branch, Corps of Engrs., Walla Walla,
   Wash.
2. Chief, Hydraulic Design Section, Corps of Engrs., Walla Walla, Wash.
3. Chief, Structural Design Section, Corps of Engrs., Walla Walla, Wash.
4. Chief, Hydro-Electric Design Branch, Corps of Engrs., North Pacific Div.,
   Portland, Ore.
5. "Engineering Advancements at McNary Project" by Louis E. Rydell and G.
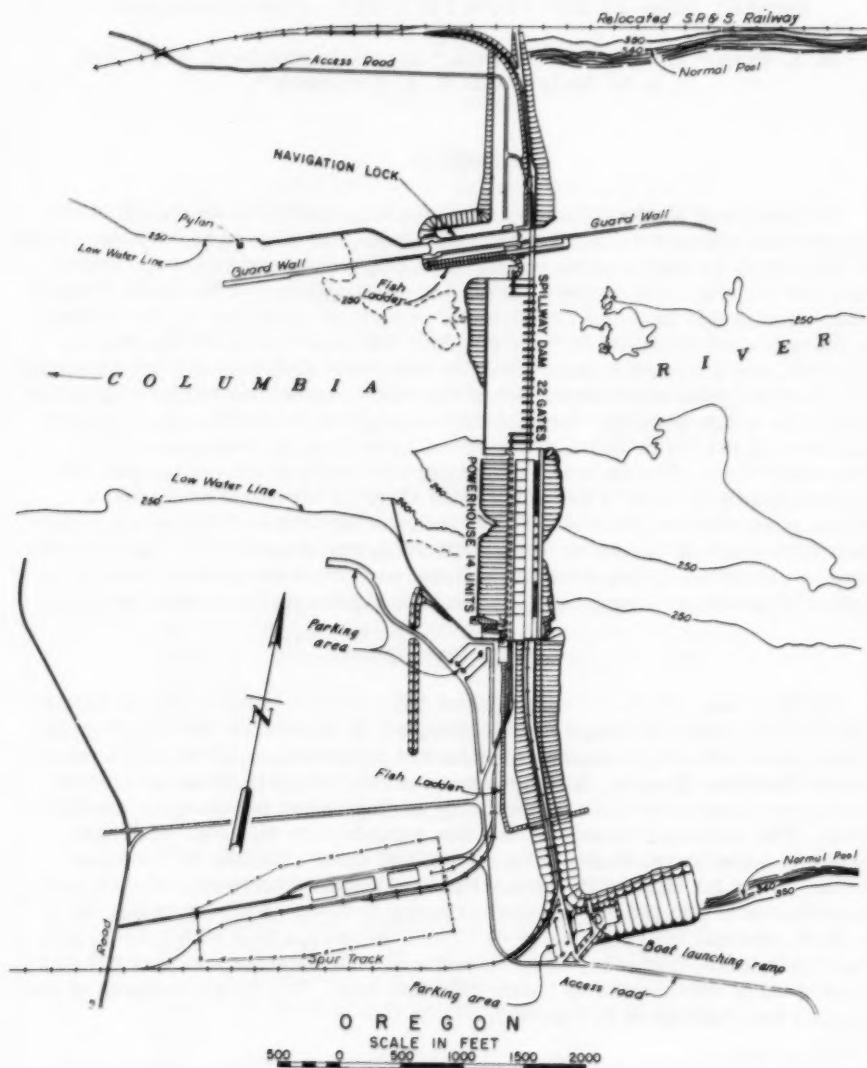   H. Von Gunten.

FIG. 1. - GENERAL PLAN

Topography and Regional Geology

McNary Dam is located in a 6000-foot wide shallow canyon in the Columbia River basaltic plateau immediately below Umatilla Rapids. The plateau rises quite abruptly to a height of 500 feet above the river on the Washington side and less abruptly to a height of 300 feet on the Oregon side. The river channel occupies about 2000 feet of the central portion of the valley. At the dam the maximum water depth at low water is about 40 feet near the left side of the channel, but depths increase to a maximum of 100 feet approximately 1000 feet below a portion of the structure.

The dam is located in the trough of a gentle downwarp within the Umatilla Basin, a subdivision of the Columbia Plateau physiographic province. The Columbia Plateau is one of the largest areas of basalt flows in the world, and the total thickness of the flows at the site has been estimated to be 3000 feet. Individual flows vary up to more than 150 feet thick. The characteristics of every flow vary considerably, but typically the top and bottom of the flows are vesicular or scoriaceous and the central portions exhibit vertical columnar joints and fairly regular horizontal to angular fractures. The contacts between flows range from tightly sealed to open. Disconformities are common and interbeds of sedimentary materials deposited between lava accumulations are observed occasionally.

Geology of the Site

The general location of the dam was dictated by the navigation requirement that it be located immediately below Umatilla Rapids and at the approximate head of the planned John Day Reservoir. The exact location was further dictated by an extremely deep "pothole" in the river bed just below the selected site and the presence of a known major fault crossing the valley about a mile downstream.

Prior to commencement of construction the site was explored by 168 NX, BX, 4-inch, 6-inch and 8-inch diameter drill holes, together with 141 probings, 25 test pits, 3 36-40 inch diameter calyx holes and 46 seismic lines. The first 24 drill holes were made by the Bureau of Reclamation in 1923-24. Corps of Engineers explorations commenced in 1938 and continued intermittently until 1948, gradually increasing in scope and coverage as the design of the dam progressed through preliminary investigation to the final design stage.

A geologic profile and section is shown in Figure 2. Bedrock at the site consists of five pertinent members of the Columbia River basalt formation; locally designated as the basal basalt flow, sedimentary interbed, main basalt flow, flow breccia bed and the upper basalt flow. The overburden consists of talus, terrace deposits and recent alluvium. The characteristics of each type of material are described below.

The basal basalt flow is not exposed on the surface at the site. The upper portion has variable porosity and is generally closely fractured. Most of the explorations indicated that a zone of 1 to 15 foot thickness in the upper portion carries water under artesian pressure. However, this zone was not prevalent in all cases.

The interbed resting on the basal basalt consists of sedimentary deposits laid down during an erosional and depositional cycle between basalt flows. It is not exposed at the surface at the site nor penetrated by foundation excavation. The explored thickness varied between 29 and 64 feet, and in general the material grades from an upper medium hard tuffaceous silt and clay through
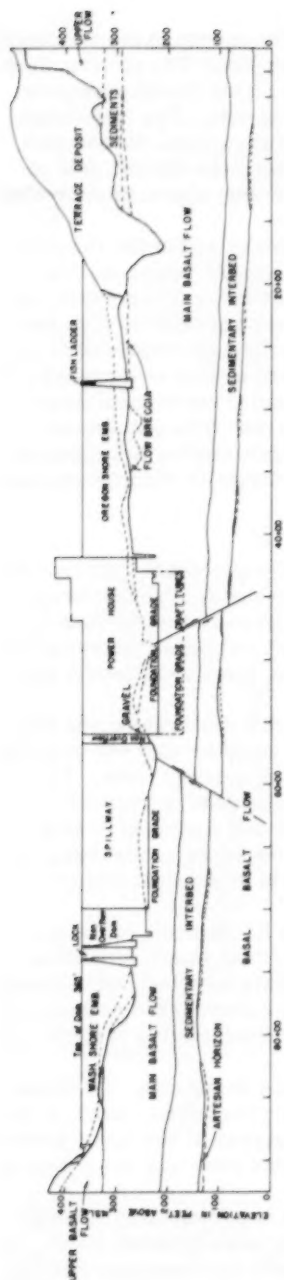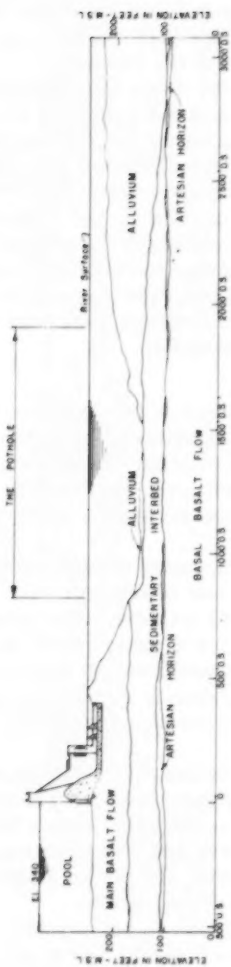
FIG. 2. – GEOLOGIC PROFILE AND SECTION

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indurated sandy silt and fine sandstone to a basal soft pebbly sandstone. Pervious materials were encountered in only one exploration near the base of the deposit.

The main basalt flow, which forms the immediate foundation of the concrete structures, rests on the interbed and has a maximum measured thickness of 130 feet. It is primarily composed of hard massively-jointed rock with typical columnar structure. However, the explorations revealed some areas of closer fracture and high vesicularity near the top of the flow, isolated vesicular areas in the central portions and many highly fractured areas near the lower contact. In general, pressure tests and grouting indicate it to be satisfactorily tight except at the lower flow contact where several explorations show an aquifer having no artesian pressure.

The flow breccia has a maximum thickness of 60 feet but in general it had been eroded completely away in the channel section of the river. It consists of angular fragments of vesicular to dense basalt ranging in size from less than an inch to several feet, all inclosed in a matrix of soft to indurated clay-like material. Its origin is problematical, but the irregular orientation of the basalt fragments and stringers appear to indicate that it is derived from partial chemical alteration of volcanic debris originating as a mud flow. It is very impervious and of insufficient bearing strength for the concrete structures.

The upper basalt flow exists in the foundation only at the north end of the Washington shore fill. It is a hard massive closely jointed rock similar to the main basalt flow.

The talus and alluvium at the site are local in extent, generally thin, and were removed during structural and earth fill core trench foundation excavation. The left abutment terrace deposit, on the other hand, had considerable effect on the project. This overburden is only 20 feet deep 500 feet riverward of the Oregon end of the earthfill, but from there bedrock surface drops off to about 150 feet deep beneath the south end of the fill. Bedrock again rises to the surface 1600 feet inland from the end of the dam. This mass of pervious deposits is apparently of alluvial origin, and is made up of sandy and silty sandy gravels up to generally a six-inch maximum size. The permeability coefficient was found to be extremely variable and, where it could be measured, values of 0.0006 to 10.0 feet per minute were obtained. Some of the open-work gravel strata could not be tested but it is possible their permeability coefficients were at least 100 feet per minute. In fact one field report during test pitting operations stated that a draft sufficient to blow out a match was observed in one of the test pits. Another complicating factor was the lack of medium sand in the gradation of the sandy gravel which tended to allow piping out of fine sand when small hydraulic heads were imposed. An alleviating factor for many of the potential difficulties was the lenticular structure and foreset bedding of the terrace, tending to decrease flow through the abutment, but also tending to make rational analysis of potential seepage subject to inaccuracy.

Concrete Aggregates

Construction of the dam was facilitated by unusually fortunate location of gravel and sand aggregate materials. During early phases of the work before haulage across the river was feasible, aggregates were obtainable from the Berrian Island deposit about 4-1/2 miles upstream from the dam on the Washington side and from Mission Beach, a mile downstream on the Oregon side. As soon as mixed concrete could be carried across the narrowing gap or over the partially completed dam the Mission Beach aggregate could be used alone,

thus concentrating concreting operations and shortening the haul. The necessary amounts of blend sand of the Nos. 16 and 30 screen sizes were obtainable within a few miles or could be manufactured from the other aggregates. It was determined early in the design stage that crushed basalt aggregate could not compete economically with the natural gravel sources.

The Berrian Island gravel consisted of subrounded to angular material with a lithologic composition of 39% basalt, 20% granite, 20% hornfels and argillite, 18% quartzite, 2% andesite and rhyolite and 1% siliceous metabreccia. The sand was hard subangular material with a lithologic composition of 66% granitics, 24% basalt, 6% andesite and 4% quartzite. Unstable silica minerals amounted to 3% and 4% in the gravel and sand, respectively.

The Mission Beach gravel consisted of subrounded moderately weathered material with a variable lithologic composition of 41-62% basalt, 9-30% granite, 7-15% quartzite and the remainder of metamorphics and andesite. In general, the percentage of basalt decreased and granite increased in a landward direction. The sand was composed of 60% granitics, 20% basalt and the remainder quartzite, andesite and volcanic glass. About 3% of both gravel and sand consisted of unstable silica.

The gravel in both pits was well graded from a maximum size of six inches, but the sand was deficient in the Nos. 16 and 30 screen sizes and had an excess of Nos. 50 and 100 sizes. Test results of both coarse aggregates showed 0.5 to 0.8 percent absorption, 2 percent loss in 5 cycles in the magnesium sulphate soundness test, 15 percent loss in 500 cycles of the Los Angeles abrasion test and an SC-RC ratio of 1.5 to 1.8. Tests of fine aggregates showed 1 to 2 percent absorption, 3 to 8 percent loss in 5 cycles of the soundness test, approximately 110% of standard for the 28-day mortar compressive strength and an SC-RC ratio of 0.9 to 1.5.

Both sources of sand and gravel were found to be non-reactive when tested by mortar bar expansion tests but were indicated to be deleteriously reactive by chemical reactivity tests. To further check the suitability of concrete constructed of aggregates from the similar terrace deposits in the vicinity, a comprehensive investigation of all known bridges and other concrete structures constructed during the 30 previous years indicated no signs of deleterious alkali-aggregate reactivity or weathering. Although the preponderance of the tests and service records indicated suitability, it was considered desirable to require the use of low alkali cement.

Embankment Materials

The primary source of fill materials for the abutment embankments was the hard durable basalt rock excavated for the structure foundations. However, considerable quantities of impervious fill and transition materials were necessary for control of seepage.

The impervious core material consisted of sandy silt obtainable from borrow pits on each side of the river. The many hundreds of tests performed during design and construction phases of the project indicate the following general properties of the impervious fill:

Gradation
Passing No. 50 sieve	95-100 percent
Passing No. 200 sieve	50- 85 percent
Less than 0.005 mm.	5- 20 percent
Compacted dry density	100-115 pounds per cubic foot
Compacted water content	15- 20 percent of dry weight

Compacted shear strength
   Tangent of angle of internal
      friction   0.60-0.70
   Cohesion   0 - 1.0 tons per sq. ft.
   Permeability coefficient 1×10^{-4} - 1×10^{-7} ft. per min.

Filter materials for the transition zone between the impervious core and rock fills were obtainable by various blends of local sand and gravel and from spalls of rock excavation.

Effect of Foundations and Materials on Dam Layout

After the general size and location of any project have been determined from economic considerations, preliminary foundation investigations, surveys and materials studies, it is necessary to determine the best safe layout and design adapted to the foundations and making most economical use of available construction materials.

At McNary the location of the site was dictated within relatively narrow limits as described above. The topography of the site necessitated that the navigation lock, spillway and powerhouse structures be located in or near the original channel area. It took little study to determine that earth abutment sections at either end of the concrete structures were considerably more economical than concrete non-overflow structures in these areas, partially because of the availability of the large amounts of embankment rock from excavation for concrete structures.

Three different schemes for layout of the concrete structures were considered. For all schemes, the foundation explorations indicated the desirability of founding the structures on the main basalt flow well above the sedimentary interbed. Two of the schemes included location of the powerhouse on the Washington side of the channel. The adopted plan provides for the powerhouse on the Oregon side to take maximum advantage of a southward dip in top surface of the interbed. The navigation lock was placed on the Washington shore because of the more favorable approach conditions and to prevent restriction of future extension of the powerhouse if and when such extension might be desirable.

The adopted structure layout provides a minimum depth of 25 feet of basalt below the bottom of the draft tubes, approximately 50 feet below the bottom of spillway stilling basin slab, and about 70 feet below the bottom of the lock walls.

The downstream Washington abutment embankment is constructed upon the talus and terrace deposits in that area but the upstream rock fill and core is extended down to the relatively small depth necessary to form a positive tie-in with the main basalt or upper basalt layer. The Oregon abutment embankment from the powerhouse southward to the point where the basalt surface falls abruptly away was founded on natural gravel deposits except in the core trench area where excavation was to bedrock. The remainder of the Oregon abutment fill was constructed on the terrace deposits, making adequate provisions for decreasing and controlling seepage.

Several of the specific problems relating to the foundations are described below. In general, nature provided a good site that presented few unexpected difficulties.

Downstream Erosion

The so-called pothole located about 500 feet below the end sill of the spillway was the subject of considerable investigation during design. This depression is some 300 to 500 feet wide and generally bottoms in the interbed

although it has presumably eroded entirely through that bed further down-stream where the strata rise in a structural warp. It is probably a remnant of earlier waterfall activity and has been partially filled with alluvial gravel. This fill could be beneficial in protecting the bedrock and preventing erosion upstream toward the sill if it stays in place after construction of the dam. Hydraulic model studies were not much help in determining if the bed rock would or would not erode and the final decision had to be based on engineering judgment. It was decided that expenditures for extensive preventive measures would not be justifiable initially in view of the doubt as to whether danger to structural foundations actually existed, and because corrective measures could be taken at a later date to insure the safety of the structures if measurements indicate there is a real problem. Soundings in the area were virtually impossible because of high turbulence during construction, but they are to be accomplished during the low water period early in 1955, after six units are on the line distributing the river flow across the entire channel.

Interbed and Artesian Zone

Since all structures are located well above the interbed, and that stratum has sufficient strength to support the imposed loads, the prime concern during design was to ascertain that it would not pipe out under the combination of reservoir head and the heads in the artesian zone immediately beneath the interbed. To study this problem, comprehensive investigations were made including both conventional and 36-inch exploratory holes, test pits, determinations of piezometric heads, percolation tests, pumping drawdown tests and experiments to determine if it was capable of being grouted if necessary. Of the 30 explorations penetrating the artesian horizon, only 22 encountered artesian pressures, indicating that the zone is not present under the whole foundation. It was also found that 70 percent of the percolation was in the upper 0.5 foot of the basal basalt flow, adjacent to the contact. Water temperature tests showed the artesian water to be $60^{\circ}F$ as compared with river temperatures of $44^{\circ}F$ but the pH and hardness were identical for both. The investigations determined that the amount of artesian head varied considerably from hole to hole and with river level elevations. During peak flows in 1949, the piezometric head averaged about five feet above the river surface at the site, while during low water periods heads varied from 0 to 20 feet above the river surface.

Pumping tests were performed in a 36-inch calyx hole at the top of interbed and at the interbed-artesian zone contact. The upper test was performed at 610 gpm for 3.25 hours and the lower test at 163 gpm for 6 hours. Neither test showed any tendency of piping of the interbed materials. It was, therefore, concluded they would not pipe under natural conditions.

Presuming a connection between the pool and artesian aquifer, a check was made to determine if hydraulic pressures in the artesian horizon after the pool was raised would be sufficient to cause blowout of the apron or rock downstream from the apron. Using very conservative assumptions, it was found that the weight of basalt and interbed were sufficient to resist such tendencies.

Foundation Grouting and Drainage

The test grouting program indicated that the more pervious portions of the interbed could not be grouted with very thin cement grout and highest feasible pressures. On the other hand, it was found that the artesian horizon could be successfully grouted, if necessary, at 50 to 350 psi, water cement ratios varying from 0.9 to 5.0, by volume, and grout holes on 5-foot centers. However,

because of the conclusion that piping was not a problem, grouting of this zone was not considered necessary for safety of the dam.

The design of the structures, which is discussed elsewhere in this paper, is based on reduction of uplift resulting from impervious foundation conditions and relief drainage near the upstream toe. The pressure tests in the drill holes in the main basalt layer on which all structures are founded indicated that this basalt is generally tight but there are occasional areas and zones of vesicular material and unfilled joints. For that reason, it was considered necessary to provide a continuous single grout curtain and drainage under all of the concrete structures. The grout curtain was designed to extend to top of interbed and the drain holes were to extend to 1/3 of that depth. A description of actual grouting operations and observations of results obtained will be presented in a future paper.

The cutoff for the Oregon embankment is principally founded on flow breccia, and there was considerable discussion as to whether grouting would be necessary. A test program and actual construction grouting indicated low takes and dubious necessity except in the vicinity of the tie-in with the powerhouse. Tests of the Washington Shore embankment foundation rock indicated the desirability of providing a shallow grout curtain under landward portions of the embankment grading into and connecting to the deeper curtain under the lock structure.

Embankment Design

The design of the two embankments was simplified by generally good foundation conditions and availability of more than sufficient quantities of good rock from structural excavation. As shown in Figure 3, they each consist of a rock shell, a relatively narrow central core, appropriate transition zones, and three feet of selected rock revetment. Except for the extreme south end, the core and filter zones extend through the alluvial and terrace deposits to bed rock. Outer slopes are 1 vertical on 1-1/2 horizontal upstream, and variable downstream depending on quantity of available rock and
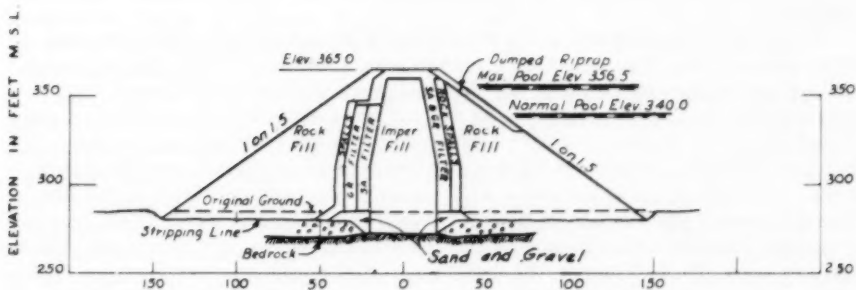
FIG. 3 - EMBANKMENT SECTION, LEFT ABUTMENT

layout requirements. The steepest downstream portion is adjacent to the powerhouse, where a 1 on 1.32 slope was necessitated by access facilities to the upper and lower portions of the powerhouse.

The upstream transition materials consisted of a natural sand and gravel zone adjacent to the core and a spall zone. The three downstream filters were sand, gravel and spalls, respectively. In the Oregon Shore embankment the zones were designed to be eight feet minimum width to allow use of natural

materials with a minimum of processing and permit ease of placement. Washington Shore embankment filter zones were reduced to 3 feet width because lack of suitable natural gravel required use of more expensive processed gravel. The maximum filter ratios (15% size of coarser material divided by 85% size of finer material) were 5 for the sand and core, 10 for gravel and sand, and 5 for spalls and gravel.

Shearing strengths used in design (tan ϕ) were 1.0 for rock fill, 0.6 for impervious core and 0.8 for foundation gravels. No cohesion was assumed for the core material. Using these values and appropriate unit weights, minimum safety factors for the embankment as a whole were of the order of 1.5 for the steady seepage case. Consideration of rapid drawdown was not necessary because such a condition would not occur under normal conditions and because of the free draining nature of the upstream shell. Minimum safety factors for the outer slopes are 1.5 upstream and 1.3 downstream.

The tests of the core materials indicated that settlement due to consolidation would be essentially complete during the construction period. Some readjustment of the upstream rock shell was to be expected when the pool was raised, and to reduce this movement the rock was sluiced with small amounts of water at low pressure. Because placement was in relatively thin lifts, and compaction was obtained from construction equipment, it was not considered necessary to provide high-pressure sluicing as is generally accepted practice for high rock fills.

Seepage Control in Left Abutment

The nature of the terrace deposits in the Oregon abutment has been described above. Flow net analyses based on permeability factors from the field tests and averaging 1 foot per minute, indicated a total seepage of approximately 70 c.f.s. through the abutment if the embankment cutoff were stopped where the top of bedrock started to drop off toward the abutment. Although the amount of seepage loss was not necessarily critical when compared with the great flows of Columbia River, control was mandatory because of the piping proclivities of the material and the necessity for maintaining the working area downstream from the dam free from uncontrolled seepage discharges.

A full cutoff to bedrock as an extension to the south end of the dam was first considered. Excavation of a trench with 1 vertical on 1-1/2 horizontal slopes and maximum depth of 200 feet, together with construction of what amounted to an underground dam in the cut, was estimated to cost 3-1/4 million dollars. A 6'-thick concrete cutoff constructed by tunnel and stope methods was then considered and found feasible at approximately the same cost. A combination of the two, using an open cut and earth fill for the 900-foot riverward section where bedrock was below original water table, and a concrete cutoff for the remaining 900 feet, was found to cost about one million dollars less. In both cases, the concrete cutoff would only extend from bedrock to five feet above normal pool.

Concurrent studies were made to determine the feasibility and effectiveness of a cutoff and blanket extending upstream perpendicular to the dam axis coupled with a drainage collection system downstream from the axis. The general scheme is shown in figure 4. Flow net analyses were made for various lengths of blanket and it was found that seepage would be about halved with a blanket 1000 feet long at a cost of $1,350,000. This plan was approved by the board of consultants and was adopted for construction. This method still allows seepage through the underground valley paralleling the abutment, but lengthens the path and furnishes complete control of the seepage water.

Field observations since completion will be reported in a future paper. In summary, the solution was found to be completely effective, although piezometer observations and seepage measurements show that approximately one-half or more of the seepage at normal pool is evidently flowing landward of the drain and is not traceable at the present time.
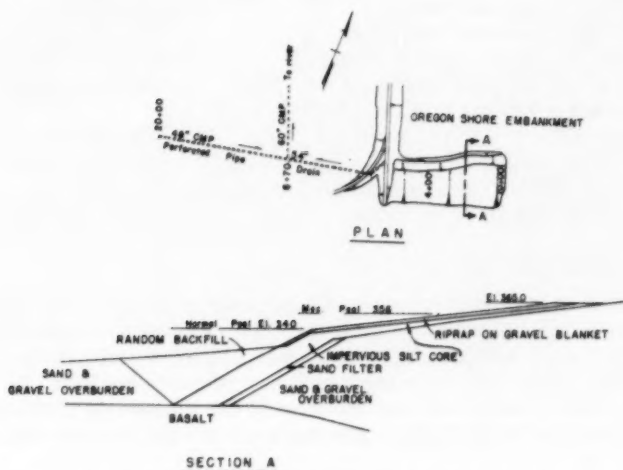

FIG. 4. - UPSTREAM BLANKET, LEFT ABUTMENT

Basic Concrete Design Factors

The comprehensive aggregate investigations were used to determine specifications for concrete construction and control joint layout. Details of construction such as mix designs and placement temperature control will be covered in a later paper. Of the approximate 1,500,000 cubic yards of concrete used, about 54 percent is mass concrete in the relatively low lock walls, non-overflow dam and the spillway section which are 125, 110 and 81 feet high, respectively.

Low alkali Type II cement was specified throughout. An air entraining admixture was also used to increase workability and enhance durability. For interior mass concrete, 0.60 barrel of cement per cubic yard was generally required, but for exterior mass concrete and during cold-weather placement this ratio was 0.75 to 1.2 barrels per cubic yard, depending on the mix. Structural concrete varied between 1.25 and 1.50 barrels per cubic yard.

Thermal studies indicated that five-foot lifts at four-day intervals would be satisfactory. Due to the shapes of the concrete sections, most of the mass concrete is less than 50 feet from an exposed face, and transverse joints were generally designed to have a maximum spacing not to exceed 60 feet, except in the powerhouse where 86-foot spacing was necessary to fit unit requirements.

Hydraulic Design
by G. C. Richardson

In a project such as McNary Lock and Dam, the hydraulic features are generally considered to be of basic importance, with the structural and other

features designed to produce the desired hydraulic effects. As a result of studies discussed in the first paper in this series the following general criteria were established for the hydraulic design of the project:

a. Normal pool elevation   340.0 ft., m.s.l.
b. Maximum drawdown   5.0 ft.
c. Maximum pool elevation   356.5 ft., m.s.l.
d. Spillway design discharge   2,200,000 c.f.s.
   Equivalent tailwater elevation   303.5 ft., m.s.l.
e. Maximum flood of record (1894)   1,200,000 c.f.s.
   Equivalent tailwater elevation   286.8 ft., m.s.l.
f. Minimum monthly regulated discharge
      with present upstream reservoirs   66,000 c.f.s.
   Equivalent tailwater elevation   250.0 ft., m.s.l.
g. Minimum discharge assumed for navigation   43,000 c.f.s.
   Equivalent tailwater elevation   248.0 ft., m.s.l.

Hydraulic Model Studies

In common with most large multiple-purpose projects built within recent years it was found advisable to study many of the features of McNary Dam by means of hydraulic models. Only those features were studied which it was considered were not subject to rigid analysis by other methods. Eight different models were constructed and operated at the Bonneville Hydraulic Laboratory, at a total cost of $875,000. The eight models are briefly described as follows:

1. General Model.—Constructed to a scale of 1:100 it reproduced 3.7 miles of the Columbia River at the dam site. See Fig. 5. It was used to determine the most desirable alignment of powerhouse and spillway, the details of powerhouse tailrace, the type and extent of navigation lock approach guardwalls, entrance and exit conditions for the fishways and flow conditions prevailing during various construction stages. One of the problems investigated was the closure of the second step cofferdam across the narrow and deep Oregon channel. The original plan of closure, involving the use of timber cribs floated into place, was shown to be impractical because of the high hawser pulls, some of which exceeded 2,500,000 pounds. This led to the selection of the method of closure by dumped 12-ton concrete tetrahedrons.[6]

2. Spillway Model.—Three spillway bays were reproduced to a scale of 1:36 in a glass-sided flume. Details of the ogee crest, piers, gates, gate slots, baffles and stilling basin were studied and determined. In general it was found that the features were adequate as designed.

3. Spillway Gate Model.—It reproduced at a scale of 1:10 all features of the prototype gate. The purpose of this model was to study in greater detail the pressure conditions on the crest, piers and gates, flow conditions at partial gate openings, with both split gate and undershot operation, venting requirements beneath the nappe issuing between the gate leaves and discharge rating of the gates.

4. Navigation Lock Model.—This model reproduced at a scale of 1:25 a portion of the upstream approach channel, the lock chamber inclusive of the

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6. Report on the Closure of the Second-Step Cofferdam, McNary Lock and Dam, July 1951, by Walla Walla District, C. of E.
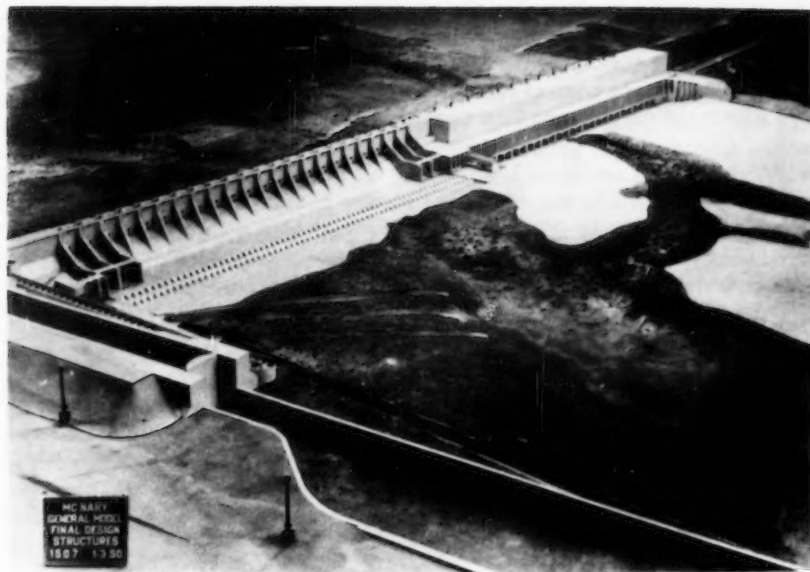
FIG. 5. - GENERAL MODEL

complete filling and emptying system, and about 300 feet of the downstream approach channel. Data were obtained on the intensity of hawser pull developed by barges moored within the lock chamber, velocities and pressures within the hydraulic system and water surface elevations in the forebay, lock chamber and tailbay throughout the period of filling and emptying. The main purpose of the tests was to obtain comparable data on several methods of lock filling and emptying and to develop details of the most desirable system. The result of these tests was the adoption of bottom-laterals for filling and emptying the lock.

5. Navigation Lock Control Valve Model.—To supplement the lock model a single control valve was constructed to a scale of 1:20, together with a portion of the culvert upstream and downstream. Forebay and tailbay tanks permitted the operation of the valve under any desired constant head and pool elevation. The purpose of this model was to study various designs of tainter valves by comparing pressures, valve loading and vibration.

6. Fishladder Model.—A portion of the Washington shore fishladder was reproduced to a scale of 1:16, including the 180° bend, twelve pools upstream and six pools downstream therefrom, as shown in Fig. 6. Investigations were made of flow conditions within the ladder pools, with varying heads, ladder widths, slopes and depths, weir shapes and spacing, and submerged orifice designs and location.

7. Fishladder Diffuser Model.—One typical diffusion chamber and the two adjacent pools were reproduced to a scale of 1:10. The purpose of the model was to develop a design for the diffusion chamber floor and expansion chamber leading from supply conduits thereto. This model was later supplemented by additional tests on the piping leading to the diffusion chamber to determine pressures and cavitational possibilities at critical points.

8. North Shore Fishladder Diffuser Model.—This model represented at a scale of 1:16 a portion of the Washington shore fishladder extending from the fish entrance 332 feet upstream, including six diffusion chambers of differing sizes, and the supply conduit from head water. Studies were made of pressures within the conduit and flow conditions in the ladder.

Spillway

General

The general features of the spillway are well described in the first paper of this series. The following discussion therefore is limited to several points of design which were not previously mentioned but are of special interest.

Crest Shape

Within recent years an attempt has been made by the Corps of Engineers to standardize the design of spillway ogee crest shapes. A crest shape for high dams with vertical upstream faces has been adopted where the depth of approach measured below the spillway crest is greater than the design head. The equation for the shape is $X^{1.85} = 2H_D^{0.85}Y$. At McNary, however, where
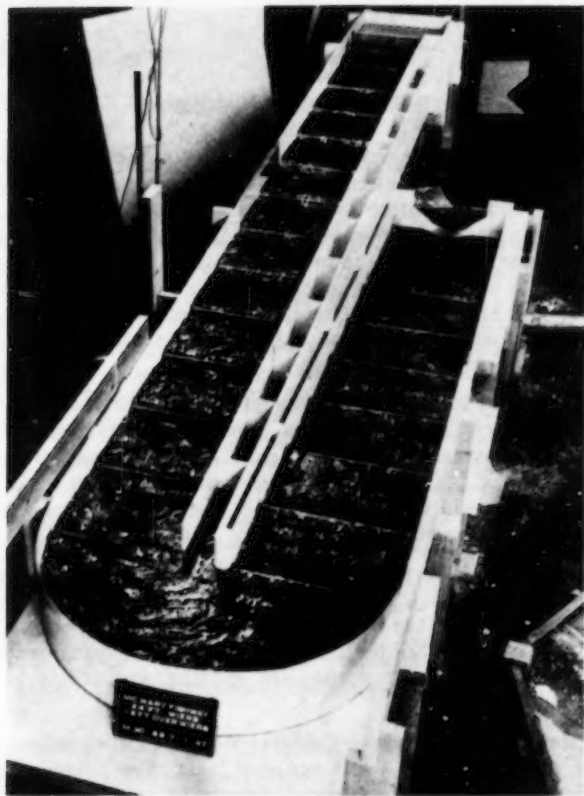

FIG. 6 - FISHLADDER MODEL

the approach depth is but 45 feet, compared to the design head of 65.5 feet, the recommended procedure requires that the crest follow the shape of the under-nappe from a sharp crested weir, based on data developed by the Bureau of Reclamation.[7] A curve was fitted to the points thus determined, having the equation $X^{1.84} = 66.635Y$ for the ogee below the crest. The design head was taken as 65.5 feet, which is that required for the maximum spillway design discharge of 2,200,000 c.f.s. A section through the spillway is shown on Fig. 7. Preliminary studies were made on the use of a so-called under-designed crest based on a design head of about three-quarters of the maximum head, to obtain the advantages of higher discharge coefficients and less material in the structures. However, too little information was available at the time (1945) to assure the safety of such a design. Today, however, the use of under-designed crests is accepted practice in many of the dams being constructed by the Corps, such as Pine Flat on Kings River, California, and The Dalles Dam on the Columbia River, Oregon. Model studies of the adopted crest shape for McNary revealed no negative pressures on the crest for either free flow or gated operation. The results of these studies were verified in the prototype in June-July 1954 by pressure measurements taken at piezometers embedded in one spillway bay. Inspection of the test bay afte.' one flood season's operation revealed no sign of significant erosion or cavitation of the concrete in the ogee or the piers.

Discharge Capacity

Prior to model tests the proposed spillway design required 24 50-foot bays to pass the design discharge at the maximum allowable pool. This capacity was based on the standard equation: $Q = C (L-KnH) H^{3/2}$ studies, however, indicated that the pier contraction coefficient, K, reduced as the head increased, actually becoming negative (-0.0033) at the design head, thus appreciably increasing the discharge beyond that computed. The coefficient, C, was checked out in the model at 3.85, rather than 3.89. Variations in discharge and coefficients are plotted in Fig. 8. This increased capacity made possible a revision of the original design in one of three ways, first, by reducing the number of bays from 24 to 22, or, second, by lowering the maximum pool by 3.6 feet, or third, by raising the crest elevation and decreasing the gate height by 3.75 feet. Decreasing the number of bays to 22 was found to be the most advantageous and was adopted. The question of negative pier coefficients is an interesting one, but space does not permit further discussion of it in this paper.[8]

Pier Noses

Model studies were conducted on several pier shapes, of which the airfoil was found most efficient, in a form approximated by compound circular arcs. Later information has shown that a simpler semi-elliptical pier nose can be substituted for the airfoil with no loss in efficiency.

Stilling Basin

The stilling basin is a conventional hydraulic jump type with two rows of baffles and an end sill. Recommended design practice in the Corps of Engineers for this type of basin generally requires a length equal to 3.0 d_2 and a

7. "Studies of Crests for Overfall Dams," Bureau of Reclamation, Boulder Canyon Project, Part VI-Hydraulic Investigations Bulletin 3.
8. "Hydraulic Models as an Aid to the Development of Design Criteria," Bulletin 37, Waterways Experiment Station, Corps of Engineers, Vicksburg, Miss.
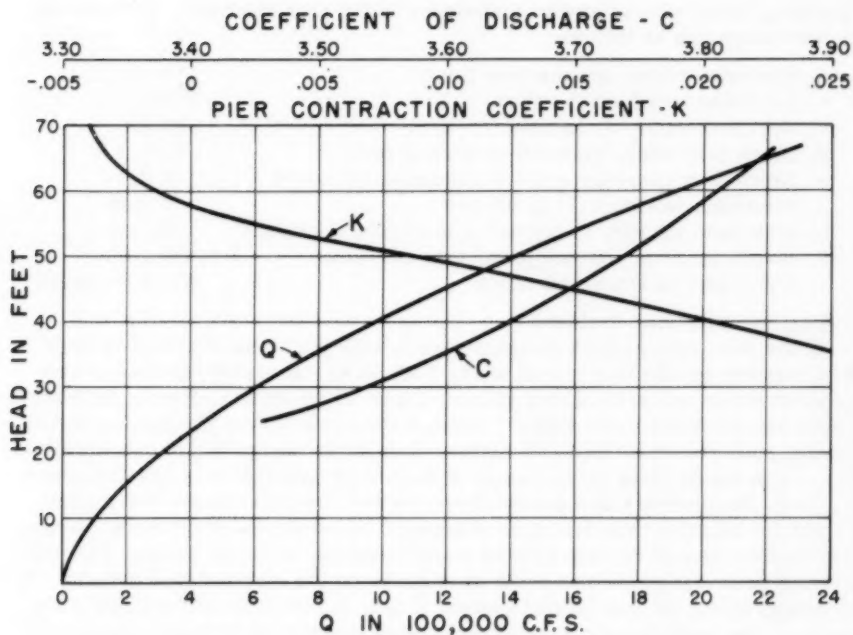
FIG. 7. - SPILLWAY AND STILLING BASIN

787-16

COEFFICIENT OF DISCHARGE - C

PIER CONTRACTION COEFFICIENT - K

FIG. 8. - SPILLWAY CHARACTERISTICS

floor depth below tailwater of 0.9 d_2. However, due to the extremely high concentration of flow of 1679.4 c.f.s. per foot of width and the relatively low specific energy of 128.2 feet, the Froude number is only 3.28 at the design discharge of 2,200,000 c.f.s. Since this lies in the range approaching an unstable hydraulic jump, model studies were considered necessary. As a result of these studies the length of basin was determined to be 270 feet or 3.22 d_2 and the depth, 75.5 feet or 0.9 d_2. The baffles are a semi-streamlined type, patterned after those developed for the Narrows Dam[9] stilling basin, and arranged as shown on Figure 7.

Navigation Lock

General

The task of designing a navigation lock with a single lift of 92 feet presented many problems heretofore not encountered, at least in such magnitude. At the time of design the highest lift lock in operation in this country had an 80-foot lift, at TVA's Fort Loudoun, while in France design was being developed for the 85-foot lift lock in the Donzere-Mondragon[10] project, practically concurrently with McNary. From the standpoint of hydraulic design the major problem was to design a filling and emptying system which would permit a short

9. "Model Study of Stilling Basin, Narrows Dam, Little Missouri River, Arkansas," Tech. Memo 209-1, Waterways Experiment Station.
10. "The Donzere-Mondragon power, irrigation and navigation scheme on the Rhone" by M. Henry, page 144, Hydraulique et Electricite Francaises, edited by La Houille Blanche.

operating cycle without severe turbulence in the lock chamber. Criteria set up for design are as follows:

a. Maximum filling or emptying time	17 min.
b. Net clear length of chamber	675 ft.
c. Net clear width of chamber	86 ft.
d. Depth over upper gate sill at normal pool	20 ft.
e. Depth over lower gate sill at minimum tailwater	12 ft.
f. Maximum hawser pull on barges	5 tons
g. Maximum velocity in downstream approach channel	6 f.p.s.
h. Maximum river discharge for lock operation	800,000 c.f.s.
Equivalent tailwater elevation	278.8 ft., m.s.l.

Filling and Emptying System

In the very preliminary design stages a wall-port type of filling system was considered, similar to that of the TVA locks. However, as design progressed it became evident that either bottom-longitudinal culverts, as model tested for the Willamette Falls[11] Lock, Willamette River, Oregon, or bottom-lateral culverts would be much better. The latter design had shown excellent results in model tests at the Corps of Engineers laboratory at the University of Iowa. Preliminary designs for both systems were developed and model tested for relative turbulence, as measured by lawser pulls on model barges. The bottom-lateral system proved so far superior (2.5 tons against 14.5 tons hawser pull) that no further tests were made on the longitudinal culverts. The principle of the bottom-lateral system is that of jets from ports in the side walls of the transverse culverts discharging into the intervening transverse trenches, thus dissipating energy partly by impingement on the opposite wall and partly by intermingling of the jets. It was found that the most efficient arrangement consisted of fourteen laterals, seven from each main wall culvert alternately and located in the center third of the lock chamber. See Figure 9. Each of the two main culverts are 11' x 12' in section with a tainter valve at the upper end for filling and at the lower end for emptying. Both filling and emptying is accomplished through the same laterals in the chamber floor with a second lateral system in the downstream approach channel for emptying. The size of the wall culverts was computed from the equation developed by the St. Paul District, Corps of Engineers:

$$A_c = \frac{2\,A_L\sqrt{H}}{C\,(T - k\,t_v)\,\sqrt{2g}}$$

in which

A_C = area of both culverts in square feet

A_L = horizontal area of the lock chamber, sq. ft.

H = maximum lift in feet

C = discharge coefficient

T = total filling time in seconds

k = valve-opening factor, dependent on shape of valve opening curve, = 0.708

t_v = valve-opening period in seconds

Model tests of this design established the value for C of 0.88 rather than 0.83 as assumed, with a resulting filling time of 16 min. 5 secs.

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11. "Model Study of Willamette Falls Lock," Rept. No. 8-1 Bonneville Hydraulic Laboratory.

- PLAN -

SCALE IN FEET
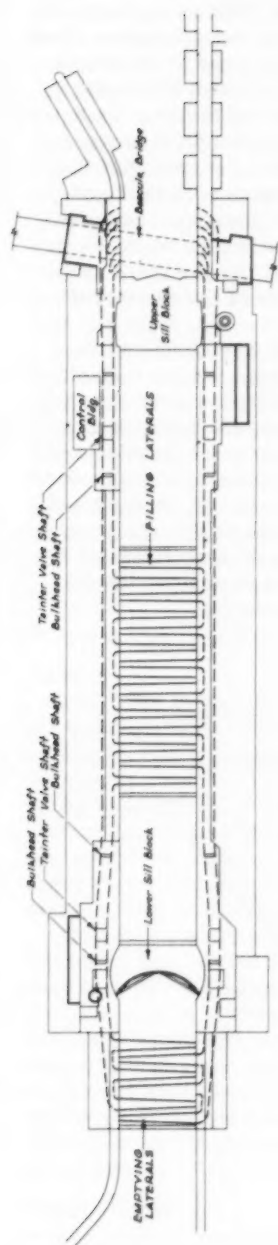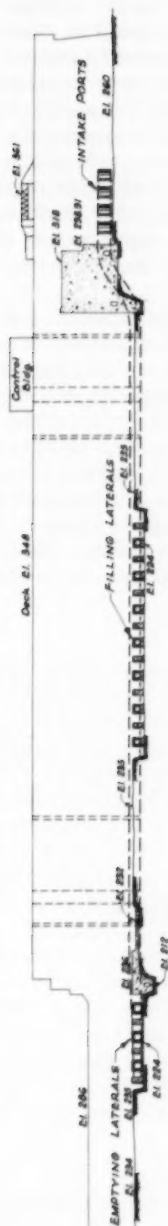
- VERTICAL SECTION -

FIG. 9. - NAVIGATION LOCK FILLING AND EMPTYING SYSTEM

Culvert Valves

The design of the culvert valves is probably the most difficult single problem encountered in the hydraulic system. Two conditions must be met. First, since the valves would be submerged 100% of the time they had to be simple and rugged, in order to reduce expensive maintenance. Second, air vents could not be permitted to relieve the negative pressures which normally occur downstream from the partly opened valve, since the introduction of large quantities of air tremendously increases the turbulence in the lock chamber. A tainter valve was selected as best meeting the requirements of the first condition. If placed in the normal position with the skin plate upstream from the trunnions the valve shaft would act as a huge air vent, which would be contrary to the requirements of the second condition. Hence the valve was reversed so that the valve sealed on the downstream side of the valve shaft. See Figure 10.

Many different valve shapes were tested in the model to determine the magnitude of the operating forces, both uplift and downpull, and the instantaneous fluctuation of these forces. The basic design consisted of a downstream skin plate supported by transverse I-beams, with I-beams for trunnion arms. Variations tested included inclosed, streamlined trunnion arms, beveled and extended gate lips and additional skin plates, both convex and concave, covering the skin plate beams. The adopted shape provides inclosed, streamlined trunnion arms, a beveled gate lip and a concave skin plate on the upstream side of the skin plate beams. This design offered the best combination of low operating forces and load fluctuation although certain other designs tested were better in either one or the other of these characteristics.
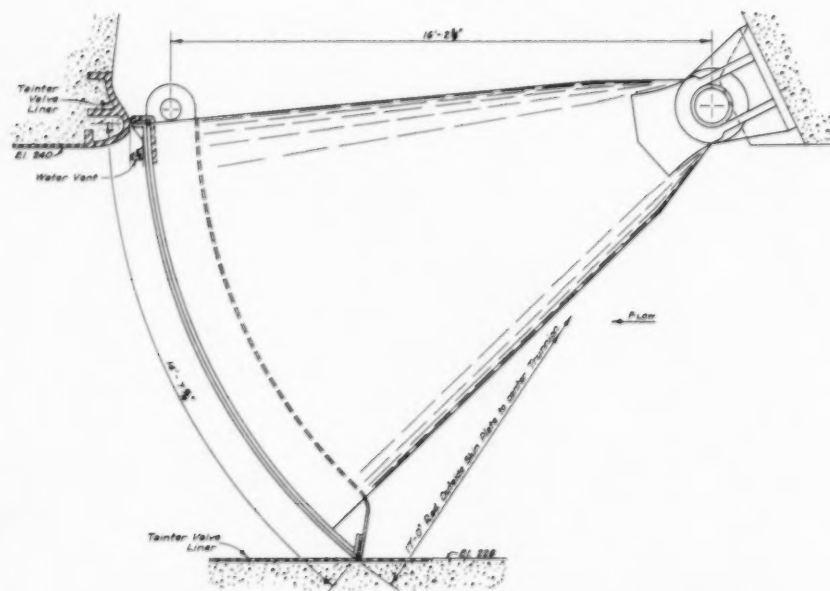

FIG. 10. - NAVIGATION LOCK TAINTER VALVE

In order to reduce the magnitude of negative pressures below the valve three measures were considered. First, the entire valve could be lowered to provide a deeper water cushion. This very positive method was ruled out on the basis of cost and because of the time of these investigations the construction of the lock was already advanced to the point that made a change impossible. Second, a so-called "water vent" could be used, which, by means of an extended lip on the upper valve seal, would permit the introduction of water from the valve shaft to the low pressure area at the culvert roof, as soon as the valve started to open. Structural limitations, as well as the status of construction, would not permit the development of this idea to its optimum, but it was possible to provide a 4-3/4 inch gap between the skin plate and the seal bulb casting on the culvert liner. Third, by lengthening the valve opening period, the water surface in the lock could be allowed to rise appreciably before the valve reached its most critical opening (about 7 feet) thus providing a greater submergence. It was found that the most effective opening schedule was a non-uniform one in which the valve was opened quickly two feet and then continued slowly but at an increasing rate until fully opened, requiring 7 and one-half minutes. Nevertheless a pressure, equivalent to -15 feet in the prototype, was recorded in the model. Since corrosion resistant steel is used on the skin plate and culvert liner this pressure was considered permissible. In addition six 12-inch diameter air vents have been provided on each valve. These are normally capped but can be opened if found necessary to prevent cavitation. Piezometers have been installed for measuring pressures in the prototype but as yet no test data are available.

Fish Passing Facilities

General

The value of and the necessity for conservation of the fishery resources of the Columbia River have been too well established to warrant further discussion here.[12] By the terms of the authorizing legislation for McNary Dam it became necessary to provide fish passing facilities that would be acceptable to the U. S. Fish and Wildlife Service and all other fishery agencies in the region—in itself no mean task. Other than at Bonneville Dam, no experience was available on the successful design and operation of fishways for passing major fish runs over a dam of comparable height. For this reason the basic design of the Bonneville ladders was accepted for McNary.

The one underlying principle which guides all fishway design is that the migrating fish tend to swim against the current. Although many other factors, such as water temperature, turbidity, stream velocity, chemical analysis and even the season of the year, appear to influence the time and rate at which fish move upstream, there are still many unknowns in the study of fish migration. Nevertheless, based on the one principle stated, successful fish ladders can be and have been built. Under a current research program sponsored by the Corps of Engineers much basic information is being collected on the mechanics of fish migration with a view towards simplification and improvement of fishways.

Design Criteria

For McNary the following hydraulic criteria were established in cooperation with the technicians of the fishery agencies:

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12. See Reference 5.

a. Fishway entrance velocity   4.0 fps
b. Min. velocity over submerged weir   2.0 fps
c. Velocity through submerged orifices   8.0 fps
d. Velocity through gross area of floor
    diffusion gratings   0.25 - 0.30 fps
e. Velocity through gross area of wall
    diffusion gratings   0.50 fps
f. Velocity in front of water intake screens   1.5 fps
g. Min. discharge from any main fish entrance   1000 cfs
h. Bottom slope of ladder   1 on 20
i. Water surface drop at each weir   12 in.
j. Head on each weir   12 in-18 in.
k. Width of ladder   30 ft.
l. Height of weirs   6 ft.
m. Minimum number of orifices per weir   2

Fish Ladders

One fish ladder is provided at each shore of the river, consisting of a con-
crete flume extending from tailwater to headwater and divided into a series of
pools by concrete weirs. See Figs. 11 and 12. Ascending fish merely have to
overcome a 12 inch fall at each weir, a feat apparently requiring little effort,
with ample resting area in the pools between, if required. The fish ladder
weirs at Bonneville have orifices in alternate ends of successive weirs,
through which most of the fish apparently prefer to swim rather than over the
weir crests. Based on this observation it was determined to make the maxi-
mum use of orifices in the McNary design. However, the effect of orifices is
to increase the turbulence in the pools with resulting reduction in effective
resting area. Numerous model studies were made of varying orifice sizes,
location, and numbers, in conjunction with weir crests of differing shapes.
The weir design finally accepted has a rounded crest and an orifice at each
end three feet from the wall with the bottom flush with the floor. On the Wash-
ington Shore these are 21" x 23" and bevelled on the upstream side. The
Oregon shore orifices are 26" x 26" and bevelled on the downstream side.
There appears to be little difference in operation between the two orifice de-
signs.

One unexpected phenomenon appeared in the prototype which did not occur
in the model because of the relatively short reach of ladder reproduced. This
was a transverse, binodal, reflected and amplified, resonant wave pattern,
which is produced by heads on the weir up to about 12 inches, and having maxi-
mum violence at about a 9-inch head. At times the height from trough to
crest is eight feet or more, the frequency being about 2.5 seconds. Fortunate-
ly the normal operating head on the weirs is from 12 to 18 inches, in which
range there is no wave action. Investigations and analyses are being made
and model studies conducted on a larger 1:10 scale model to determine more
about the nature and causes of this action. These studies are incomplete as
yet. (1954)

Fishway Entrances

Selection of the most advantageous locations for entrances is one of the
most controversial and important phases of good fish-ladder design. Obvi-
ously the most ideal conditions within the ladder are of no value if the fish
cannot be enticed into the entrance. Previous experience and observations
have revealed several general principles which were used in locating the
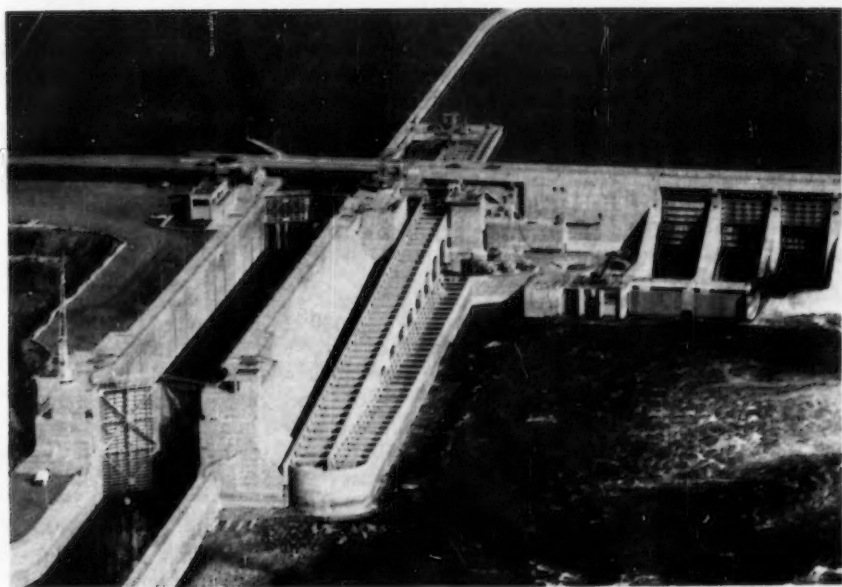entrances. The migrating fish appear to move along the shore line or at the

FIG. 11 - NAVIGATION LOCK AND FISHLADDER

FIG. 12 - OREGON SHORE AND FISHLADDER

edges of high velocity currents rather than in the main stream. Furthermore fish are reluctant to retreat from a position to which they have advanced even though the river conditions at that point may be too severe for easy passage. It is then desirable to place a fish entrance close to high velocity currents, such as from the spillway, so that the fish can readily find conditions to their liking when blocked by the dam. Hence there are entrances on each end of the spillway and at the Oregon shore end of the powerhouse. In addition 44 entrance weirs were provided at the tailrace side of the powerhouse to enable those fish who are attracted by the flow from the draft tubes to enter the fishway collection channel extending the full length of the powerhouse and leading to the Oregon shore fishladder. The collection channel can be seen in Figs. 12, 14 and 15. All entrance weirs are automatically float controlled to rise and fall with tailwater.

Auxiliary Water Supply System

According to criteria a minimum flow of 1000 cfs is required out of the main entrance of each fish ladder to attract fish. Since only 125 to 220 cfs are flowing down the ladder, the remainder must be added at the lower end. This is done through diffusion chambers in the floor of the ladder, which serve to destroy excess energy and turbulence which might otherwise cause the fish to fight the floor gratings. The auxiliary water system also provides additional water required when rising tailwater submerges portions of the ladder, thus reducing velocities to ineffective values.

On the Washington shore all auxiliary water is taken from the forebay through four gated conduits, one of which supplies the fish lock and the others the ladder proper. The total amount of water which normally will be added varies from about 900 cfs at minimum tailwater to 3700 at tailwater for 600,000 cfs, which is the maximum design flow.

On the Oregon side, part of the auxiliary water amounting to 1260 cfs is supplied by gravity and need only be added when tailwater exceeds elevation 256. The bulk of the auxiliary water, which is required at lower tailwaters, it has been found more economical to supply by pumping. This was discussed by Rydell and Von Gunten in the first paper. The pumphouse, which is shown in the central foreground of Fig. 14, contains three 2500 cfs propeller type pumps, supplying water for the entrance at the north end of the powerhouse, the entire powerhouse fish collection channel, the Oregon shore entrance and the lower end of the ladder.

Structural Design
by H. M. Rigler

General

The structural design of the various features of McNary Dam is based on the applicable chapters of the "Engineering Manual for Civil Works" of the Office, Chief of Engineers, Department of the Army, Corps of Engineers.

The designs for the Navigation Lock, the Washington Shore Fish Ladder, the North Non-Overflow dam section and the north one-half of the Spillway Dam were prepared in the Portland District, Corps of Engineers, before the formation of the Walla Walla District in November 1948. The structural design of the powerhouse was prepared by the Hydro-Electric Design Branch of the North Pacific Division Office.

Spillway

The number and size of spillway bays, the crest shape and pier shapes

having been determined by hydraulic considerations, the approach to the structural design is greatly simplified.

To a great extent the spillway portion of the dam is merely a structure supporting the gates. The piers which support the service bridge as well as the gates are founded on and their bases are an integral part of the ogee section of the dam.

Six conditions were analyzed for stability:

1. Reservoir empty.
2. HW 340, TW 250, gates closed (Normal pool, min. low water).
3. HW 340, TW 250, gates raised.
4. HW 356.5, TW 303.5 gates raised. (Maximum Design Flood)
5. Case 2 with earthquake.
6. Case 3 with earthquake.

Earthquake acceleration - 0.1 g; t = 1 sec.

The spillway dam as designed is amply stable under any of these conditions. The maximum foundation pressures are 11,400 psf at heel (case 1) and 9,200 psf at toe (case 5). Uplift pressure used was full reservoir head on upstream face to full tailwater head on downstream face acting over 50% of the base area. In the first 13 spillway piers structural steel erection towers were used which were concreted in place in the piers. The use of these towers permitted the erection of crane and roadway girders before the concrete in the piers was completed. However, in the remaining piers it was determined that the use of these steel towers was not necessary or justified and they were omitted except for a short section at the top of the pier to provide proper bearing for the crane and roadway girders. The piers were designed as cantilevers with due consideration given to eccentric loading as with gate closed on one side and gate open on the other.

Spillway Service Bridge

The welded structural steel service bridge spans are designed to meet Navy Specification No. 12Yb. Each 55 foot span consists of two crane girders 34'-0" o.c., extending 38-6" above the roadway deck and two intermediate girders carrying a 12" reinforced concrete roadway slab with removable panels over the emergency gate slots. A sidewalk and enclosed service gallery are provided between the downstream girder and a reinforced concrete girder which is supported on brackets at each pier.

The 7' deep crane girders are double web plate girders designed to carry the 200 ton gantry crane on 55' simple spans. The loading for each crane girder is 320 kips supported on 4 wheels 3'-0" o.c. in any position. This loading is approximately equivalent to a uniform load of 11 kips per lineal foot. The maximum bending stress in the crane girders is 17,000 psi and the maximum shear in the web plates is 5,000 psi. This is rather low but the thickness of the web plates is determined by specification.

Expansion is provided at alternate piers, at the other end of the span the adjoining girders are fixed against longitudinal, horizontal movement.

Spillway Gates

The spillway gates are of the vertical lift type with a nominal opening size of 50' by 50'. The span between wheels is 58'-10". The gate consists of two leaves which may be raised separately or as a unit. The upper leaf is 27'-3" high; the lower is composed of 3 articulated segments with an aggregate height of 24'-6". Each lower segment has 2 wheels to a side while the upper leaf has 3 wheels to a side. The maximum design load on any wheel is 265

kips. The wheels are mounted on cantilever axles which are framed into the gate end girders.

Each of the three lower segments of the gates is designed as a box girder with 2 webs, the skin plate acting as the compression flange and a single plate attached to the webs as a tension flange. Intercostals in gates were used in order that a reasonable skin plate thickness could be attained. The addition of a third web plate in the lower segments to reduce plate thickness reduces the spacing to the point where fabrication clearances are critical. Comparative design showed that, on these gates, the use of intercostals produced the most economical gate, regardless of how the web spacing was juggled.

The end girder which supports the wheel axles is designed as a box, the better to withstand the torsional stresses due to the eccentric wheel load.

Unusually close tolerances were required on these gates, and since they were designed as welded structures some doubt was expressed by several fabricators whether these tolerances could be met. As it turned out, by maintaining a carefully prepared welding program not too much trouble was had and the gates were delivered and placed in use with a minimum of difficulty.

The dogging ladders are designed for an equivalent load of 325 kips which includes 100% impact. AISI 2330 steel with a yield point of 100,000 psi and B.H.N. 200-225 was specified for these ladders.

A lifting beam was chosen for raising the gates because of space limitations in the slots which made the use of separate blocks impractical. The pins are engaged by means of a mechanically operated link and rod system which, it is believed, is more positive than that of automatic hooking or link engaging. Gate slot heating is necessary in certain gates to prevent the gate from freezing fast in the severe winter weather which sometimes prevails at McNary. Heater slots have been provided at all piers but only those gates which must be used for pool regulation will be heated. The heating elements extend downward to the bottom of the upper leaf. Under the usual low winter flow in the river adequate regulation can be maintained without using the lower leaves. The power requirement is approximately 25 KW per slot, 5 KW of this being for the gate side seal heating element. The heating elements are removable and interchangeable.

Bubblers have been provided to prevent icing at the upstream face of the gates.

Navigation Locks

General

The 86 ft. by 675 ft. navigation lock with single lift of 92 ft. and its operation have been generally described in the section on Hydraulic Design.

The side walls of the navigation lock consist of gravity type monoliths. On the land side the monolith retains a rock fill for its entire length. The lock chamber is 86 ft. wide by 683.5 ft. long (nominal 675 ft.) in the clear with a maximum lift of 92 ft. from tailwater at 248 to the normal pool elevation of 340. The deck of the lock structure is at elevation 348 with a minimum width of 16 ft. and the floor of the lock chamber is at elevation 235. See figure 13.

Stability analyses were made for the following conditions:

1. Construction period—Lock dry, may or may not have rock fill on land side.
2. Normal operating condition.—HW 340, TW 248, water in lock chamber at elevation 248.
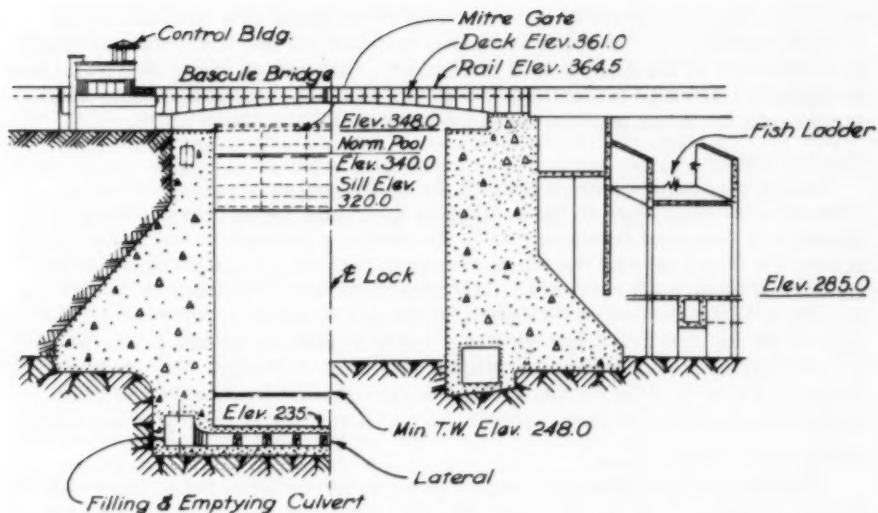2a. Case 2 with earthquake.

FIG. 13. - SECTIONAL VIEW OF NAVIGATION LOCK

3. Same as 2 except lock chamber full to elevation 340.

3a. Case 3 with earthquake.

4. <u>Maximum Operating Condition</u>—River flow = 800,000 cfs. HW 340, TW 278.8, lock chamber dry.

4a. Case 4 with earthquake.

5. <u>Maximum Flood Condition.</u>—HW 356.5, TW 303.5, water in lock elevation 303.5.

Uplift is considered as due to full hydrostatic head acting as follows:

1. Over 50% of the foundation area between the upper and lower gates.

2. Over 100% of the foundation area above the upper gate and below the lower gate.

Steel Reinforcement

Reinforcement is provided around all rooms, culverts, galleries, shafts or other openings in the lock monoliths. The amount of steel is determined by the loads acting on the section under consideration with a minimum steel area in each direction, both vertical and horizontal, of 0.0025 the cross-sectional area. Only a 12 inch thickness of concrete is considered for temperature steel where the opening is in mass concrete. All removable concrete slab covers for stop log storage pits, machinery rooms, tainter valve shafts, and other openings are designed for highway bridge loading of H 20-S 16.

Guard Walls

The 1,400 foot upstream guard wall is a gravity type with 30% openings to create a slight current from the navigation channel into the spillway forebay. A 300 kip impact load can be safely withstood by this wall with less than 60 psi tension in the concrete.

The downstream guard wall is of two types. 980 lineal feet directly downstream from the lower end of the lock is a gravity type. The 540 ft. section

of wall downstream from this gravity wall is designed as a buttressed wall on pile foundations. This was necessary because the depth of rock foundation made the cost of the gravity wall excessive. The downstream guard walls are designed for 110 kips impact load or 25 kip pull load. All guard walls both upstream and down are provided with mooring hooks and check posts.

Tainter Valves

Tainter valves are used to control the filling and emptying of the lock chamber. As described in the section on Hydraulic Design the valves are placed in a reversed position, that is the convex skin plate downstream. This places the trunnions and their anchorages in tension and the stresses in the tainter valves are all reversed from those in the usual tainter gate. With the exception of these reversed stresses in the gate members, the wrap around plate over the gate arms and the inner concave plate on the upstream face of the skin structure, there are no unusual problems of design. The tainter valves are operated by means of a hydraulic cylinder mounted in the shaft above the gate and connected directly to the lifting eye at the top of the valve.

Downstream Miter Gates

The downstream miter gate leaves are the conventional swinging arch type having a radius of 53'-6" and each leaf subtending an angle of 60º. The net width of the lock is 86 ft. and the height of the gate above the sill is 105'-11". Under load the gate acts as a 3 hinged arch with a span of 92'-8" between abutment bearings. The maximum design operating head is 92 feet at low tailwater. Two designs were prepared for this gate, one of all welded construction and one of riveted construction. In the welded gate, which was selected in alternate bidding, the stressed skin concept was used. The curved skin plate is designed for combined bending and compression stresses. The curved ribs were designed as short columns, subject to direct load. Acting as a 3 hinged arch the gate has very high safety factors for the different ribs against buckling.

The structural frame of the gate is fabricated of carbon steel. The skin plate is made of low alloy steel, elastic limit 50,000 psi, in order to reduce weight. The heat treated quoin and miter seal blocks which are subjected to maximum pressures of 26 kips per linear inch are of forged carbon steel with an elastic limit of 50,000 psi.

The gate leaves are supported at the base by a hemispherical pintle of nickel steel and at the top by a conventional gudgeon linkage which allows a precise adjustment for centering the gudgeon pin over the pintle center line. Adjustable quoin and miter seal blocks permit precise adjustment of their contact lines before the blocks are finally zinced in.

Operating Machinery

The forces required to operate the lower miter gate leaves are not subject to a rigid analysis. A program for the experimental determination of these forces was set up at the Waterways Experiment Station at Vicksburg early in 1949.

For the purpose of designing the operating machinery at McNary it was assumed that the following resisting forces must be overcome:

1. Frictional resistance of pintle and gudgeon bearings due to the weight of the gate leaf (in this case 690 kips).
2. A 0.5-foot head against the submerged portion of the leaf with tailwater at elevation 250.
3. Wind force on that portion of the gate leaf above water surface at 23 M.P.H.

Both upstream and downstream miter gates are opened and closed by the use of a strut connected to the gate at one end and to a crank at the other. This crank is mounted on a horizontal sector gear which operates through approximately 180°. The power is furnished by a 20 HP motor which drives the sector gear through a speed reducer.

The gate moves from zero with a gradually increasing motion to the mid-point of travel and a gradually decreasing speed to the end of travel which may be either the open or closed position. Appropriate brakes and limit switches for safety of operation are included in this installation.

Stop Logs

Stop logs are provided to permit unwatering of the lock for repair and maintenance purposes. Six stop logs are equipped with bearing wheels so that they may be lowered under head and used as upstream emergency bulk-heads. All other stop logs have skid type bearing plates.

The stop logs, which have a span of 88'-9", are designed as horizontal welded Pratt trusses of box shape. The web of the compression flange acts as the upstream skin plate and is thus subjected to combined stresses. Low alloy steel is used in all members to reduce weight for operational reasons.

A lifting beam and two fixed derricks, one at each end of the locks, are furnished.

Non-Overflow Dam Sections

The concrete non-overflow dam comprises two gravity sections totaling 347.9' in length; the north non-overflow section extends 254.9' between navigation lock and spillway and the south non-overflow section being 93' long between spillway and powerhouse. Top deck width at El. 361 conforms with spillway deck to provide support for 200-ton gantry crane track with rails spaced 34'-0" on centers. Aside from serving its main function as a dam its design was further complicated by various additional features outlined below. Design criteria and stability requirements are practically identical with that of the spillway.

North Non-Overflow

Five monoliths comprise the north non-overflow section. Two pits, one for storage of one spare spillway gate and the other for servicing of the gates are covered by removable deck slabs to be lifted by the spillway cranes.

A series of service rooms at various levels have been incorporated into the massive section, such as substation room, locker room, toilet facilities and septic tank. Access to these spaces is provided by an interior elevator and also by means of an outside stairway along the downstream face of the dam, with connecting galleries.

Intakes for six 6' x 8' auxiliary water supply conduits for the downstream end of the Washington shore fish ladder are also located in this section. At the upstream face, a trashrack structure with 1" bar openings has been provided. Due to the narrow openings between bars, a hydrostatic design load equivalent to 40 foot maximum differential head was imposed on the structure. Operating and control equipment such as machinery, slide gates and valves are housed in a room attached to the downstream face of the non-overflow section. The compressor room is located above the valve house. Access to these two rooms is by means of stairways. The trashrack structure and the rooms housing the compressor and valves were analyzed as rigid frames.

Passage for the 30-ft. wide Washington shore fish ladder and the 10-ft. wide fishlock exit channel are also provided through the north non-overflow

section with invert 12' below normal pool elevation. In case of spillway design flood, stop logs have been provided for closure of the two channels across the upstream face of the dam.

Downstream and contiguous to the north non-overflow section are the Washington shore fishlock and fish ladder entrances as shown in Fig. 11. The lower service deck is at El. 285 and is actually an integral part of the Washington shore fishway.

South Non-Overflow Section

The 93' long south non-overflow section is constructed in two monoliths, north and south. Although similar to the north non-overflow section in its exterior outline, the interior or the south non-overflow was subdivided for radically different purposes.

The north monolith contains a unique experimental structure, a pressure fishlock, designed to pass migratory fish from tailwater to forebay. Main components of the pressure fishlock are: (1) the approach fish ladder, (2) the fishlock pressure chamber, and (3) the exit channels.

Adjacent to the powerhouse is the south monolith which conducts the discharge from the ice and trash sluice around the north end of the powerhouse to tailwater beyond the tailrace deck. A personnel entry into the powerhouse is provided by extending the downstream service gallery through the north wall of the powerhouse at El. 346. Several rooms have been carved out in the mass section to provide storage space and to save concrete not needed for stability.

Access to the downstream service deck at El. 287 is by means of an inside stairway from top deck El. 361 and from adjacent tailrace deck at El. 287. A contraction joint 119' downstream from the construction base line separate the south non-overflow section from the intermediate fishway entrance and diffusion chamber monolith.

Oregon Shore Fish Ladder

General

The fish ladder is an open flume, sloped one foot in twenty, joining the reservoir pool and tailwater. Constructed entirely of reinforced concrete, it can be divided by location into the design condition classifications discussed herein. The ladder has an inside width of thirty feet and carries an 8 foot maximum depth of water. Construction of the ladder was divided into monolith sections of 100 ft. max. length, separated by one-half inch expansion joints. Rubber water stops were used throughout in joints subjected to water pressure on one side. A general view of the ladder is given in Fig. 12.

Ladder Sections within the Pool Area

Intake Structure

The intake structure is set on firm basalt rock at elevation 265 and extends to elevation 348. It is the intake for the pressure conduit supplying attraction water for diffusion chambers. The structure houses a submerged tainter gate to provide emergency closure of the conduit, trash racks, and a traveling screen to prevent passage of fingerlings into the pressure conduit. Stop logs are provided for closure to allow unwatering. The structure is designed to resist the loads due to unwatering in addition to standard structural considerations.

Ladder Sections on Piers in the Pool Area

These monoliths are supported by three sets of two piers. The walls of the ladder are designed as beams to span between piers, being continuous over the center piers. The walls and floor of the ladder are designed as frames (pin-connected at the top by intermediate struts) to resist lateral loads from internal or external pressure.

Unwatering the ladder channel at normal pool level results in an uplift pressure head of 16 feet on the ladder floor. Piers are designed to resist the differential uplift by dead load with a safety factor of 1.2.

End piers of adjoining monoliths are joined together near the base, with the upper portion designed to resist the forces due to expansion of monolith.

The ladder floors are designed as two-way slabs supported by the side walls and the pier bents.

Design Loads

The assumed design loads for these sections are listed below:

Live Load	- 100 psf plus machinery on operating deck.
Hydrostatic Uplift	- Full static pressure head over entire foundation base area.
Wind Pressure	- 30 psf.
Wave Pressure	- 1,000 pounds per lin. ft.
Ice Pressure	- 2,500 pounds per lin. ft.
Earthquake	- Acceleration of 0.1 gravity.

Stability Design Loading Conditions

Monoliths are designed for the loadings resulting from conbinations of conditions listed below:

Construction Condition.—Pool empty, ladder operating at 8 ft. depth (by pumping) with earthquake or with wind.

Normal Operating Condition.—Ladder operating with either earthquake, ice, or wind and wave loading. Pool El. 340.

Unwatered Condition.—Ladder channel unwatered, pool at El. 340.0.

Fish Ladder Sections thru Earth Fill Embankment

Side walls of these sections are carried vertically to the top of embankment as lighting requirement for a fish ladder will not allow enclosed sections. Wall surfaces next to the fill were sloped one horizontal to ten vertical to provide for settlement of fill to increase the seepage resistance. A bridge was required to carry the access railway and highway over the ladder. These requirements resulted in the design of the center abutment ladder section as a two story, variable moment of inertia, rigid frame, the lower deck being the fish ladder floor and the upper deck being the bridge with the walls retaining the fill.

Fish Ladder Sections Downstream Above Ground

These sections are "table" structures. The fish ladder channel spans between two pier bents with the walls designed as beams. Pier legs vary in length from a maximum of fifty feet and are set on spread footings bearing on breccia rock (allowable bearing 13K/sq. ft.).

Bridge for Powerhouse Access Railway

This is a 100 ft. long skew bridge designed of reinforced concrete and supported on five rigid frames. The frames are closed box sections encircling the fish ladder.

Fish Ladder Sections Downstream—Below Ground Level

East (Pool side) walls of these sections are designed to resist the load of saturated backfill due to possible seepage through the earth fill abutment. That portion of the wall above rock and below ground level is designed as a gravity retaining wall. The portion of wall below the top of rock was anchored into the rock by grouted anchor rods. The ladder floor is held by anchor rods into the rock to resist uplift pressures due to tailwater when the ladder is unwatered.

Pressure Conduit

The two barrel section of the conduit is designed together to resist the pool pressure when unwatered as a rigid frame. Downstream sections are designed for internal pressures. Sections containing changes in direction are designed for the unbalanced thrust.

Fish Viewing Bldg.

A suggestion by an employee of Portland District in 1948 led to the adoption of a fish viewing building with aquarium windows in the side wall of the Oregon Shore Fish Ladder. A novelty, surely, and one which would be a great attraction for the tourist laity. The consistent murkiness of the water, due to the very high percentage of solids in suspension, and bubbles due to passage of water through the weir orifices makes it very difficult to see objects for more than 2 or 3 feet from the face of the window. Also the presence of the ubiquitous lamprey, who has a peculiarly unpleasant aspect does not add to the attractiveness of this feature.

Washington Shore Fish Ladder

The fish ladder on the Washington shore is similar in design and operation to the Oregon shore ladder.

<div align="center">

Power Facilities
by R. A. Schuknecht

</div>

Selection of Type and Size of Turbines

Original studies indicated that nominal firm capability at McNary for three stages of basin development should be: 698,000 Kw, initial stage; 712,000 Kw, intermediate stage; and 875,000 Kw for the ultimate stage. Subsequent studies related to the economic height of the dam, explained briefly in the previous paper, resulted in the decision to install 980,000 Kw initially. Several sizes of both Francis-type and Kaplan-type turbines were studied to determine the most economical size, type and number of units, resulting in the selection of 14 Kaplan units of approximately 280-inch runner diameter. Contributing factors in the determination were: higher efficiencies of Kaplan units over the range of head in which firm capability is required; higher normal operating speed of Kaplan units which results in physically smaller generators and a thrust load of approximately 4,000,000 lb.; and the advantage Kaplan units have over Francis units in that fingerling salmon are believed to pass downstream through them with greater safety. There being some latitude in the permissible height and width of turbine water passages within the powerhouse structure, specifications for the turbines called for Kaplan-type units of not less than 280-inches diameter with a definite limitation placed on the elevation of the lowest point of the draft tube because of the proximity of the interbed layer beneath the foundation basalt. The turbines procured are automatically adjustable, six-blade Kaplan type with a runner diameter of 280-inches. The design head is 80-feet with a range of head of 62.0 feet to 92.0 feet. Firm capability of the plant is attained for river flows up to 800,000 c.f.s. with the

reservoir pool held to its normal elevation 340. The turbines deliver in excess of the required 111,300 horsepower at a net head of 80.0 feet, which is their rated output, and at least 93,000 horsepower at a net head of 63.0 feet. The rated speed is 85.7 r.p.m. The turbines and generators are both designed to withstand, at working stresses not exceeding 2/3 of the yield point, an overspeed of 190 r.p.m. The maximum "on cam" overspeed is 186 r.p.m., determined from model tests. The highest speed attainable, however, is approximately 217 r.p.m., also determined by model tests, resulting from a critical false blade-angle to gate-opening relationship which is possible but highly improbable. Working stresses of both turbine and generator at this maximum speed will be approximately 87.5 percent of the yield point of the materials. Maximum runaway speeds were limited as the result of model tests in that it was found entirely practicable to limit the minimum blade angle to 16 1/2 degrees[13] in lieu of the normal minimum angle of approximately 6 degrees.

The specifications under which the turbines were purchased required a guaranteed efficiency of not less than 86.0 percent at rated head and load as determined by model tests. The capacity of the prototype runner was determined from the model tests by ratios of the 3/2 powers of the heads and the squares of the runner diameters. The efficiency was determined from the efficiency of the model corrected by the Moody formula:

$$Ep = 100 - (100 - Em) \left(\frac{Dm}{Dp}\right)^{1/5}$$

where Ep = expected efficiency of the prototype in percent.
       Em = test efficiency of the model in percent.
       Dp = diameter of prototype runner.
       Dm = diameter of model runner.

The model tests indicate that turbine efficiencies up to 94 percent may be expected at best gate operation, with resultant expected plant efficiency of about 91 percent.

Turbine Setting

The specifications under which the main generators were purchased require that the maximum temperature rise of both the armature and field windings shall not exceed 60 degrees C. when delivering full rated output continuously at normal voltage, power factor and frequency. They also require that the generators be capable of delivering 115 percent rated Kva continuously at rated voltage, power factor and frequency without exceeding the allowable temperature rises for the various parts as given in the A.S.A. Standards for generators having Class B insulation. In this case, the continuous overload rating is 80,500 Kw, which coupled with a generator efficiency of 97 percent requires a 111,300 horsepower turbine. From the experience curves furnished by turbine manufacturers for a 280-inch Kaplan unit, it was found that the runner must be set at elevation 231.0 to satisfy the generator continuous overload ratings without excessive cavitation. Setting the runner at a lower elevation could not be justified, not only on account of excessive cost, but because of foundation conditions which limit the lowest point of the draft tube to elevation 180.0.

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13. See paper No. 51-A-100 "Runaway Speed of Kaplan Turbines" by G. H. Voaden, A.S.M.E. Transactions for August 1952.

Generators

The generators selected for this plant are the result of a coordinated effort with engineers of the Bonneville Power Administration to obtain units with electrical characteristics best adapted to the requirements of their transmission system. While the 70,000 Kw rating of the units was readily fixed by the normal turbine ratings, considerable latitude was possible with respect to power factor, reactance, WR^2, provision of and type of amortisseur windings and the speed of response of the exciters. The pricing policy of the electrical industry embodies standard base prices for machines, dependent upon size and speed but based on standard characteristics. The choice of machines having characteristics at variance with standard values results in additional cost depending upon the degree of deviation. In the case of the McNary generators, the Bonneville engineers desired machines of very low reactance in the interest of economic transmission of as large blocks of power as possible. In fact, the special studies they conducted on their system analyzer proved conclusively they could eliminate one 230,000 volt transmission line from the plant by virtue of the special electrical characteristics of the generators at a net saving to the Government of approximately 6 1/2 million dollars. Consequently the specifications for the McNary main generators required the following special characteristics each of which could be obtained only at extra cost in the interest of more stable transmission over fewer transmission lines:

    a. Guaranteed unsaturated direct-axis transient reactance—not more than . . . . 32% (actual value obtained = 26.3%)

    b. Short-circuit ratio, not less than 1.9 (actual value = 2.14)

    c. Nominal exciter response for main exciter—not less than 1.5

    d. Continuous connected amortisseur winding provided.

    e. Flywheel effect (WR^2) of rotating parts of generator and exciter, not less than . . . . 130,000,000 (actual value = 234,688,000) pounds-feet-squared

These generators are considered to be the largest in physical size of any made to date with a rotor diameter of 35' - 1" and an over-all diameter across the air housing of 51' - 8". Their physical dimensions roughly correspond to those of a machine having standard electrical characteristics and a rating of approximately 90,000 Kw. They are equipped with the largest thrust bearings ever built, located above the rotor and designed to withstand the combined load of the resulting weight of the rotating parts and hydraulic thrust in excess of 4,000,000 lbs. Also, in conformance with the policy of the design office for units with Kaplan turbines, two guide bearings are provided; one located above the rotor, the other located below.

Powerhouse Intake Gates

Each of the 14 main units is provided with three vertical tractor type intake gates, one for each of the three intake openings. Each gate is approximately 54-feet high and 22-feet wide and weighs about 180,000 lbs. The decision to provide a full complement of intake gates for all units is based upon the probability of and consequences resulting from a full, uncontrolled runaway of a main unit. Although the machines are designed to withstand runaway conditions, the terrific pounding within the water passages and vibration of the unit and of the structure resulting from overspeed conditions are to be avoided or eliminated as quickly as possible. In the event an overspeed is experienced due to loss of governor oil pressure or due to breakage of the wicket gate control linkages whereby governor oil pressure cannot be brought to bear in a corrective manner, the only means of controlling the runaway

machine lies in successful operation of the intake gates. In such cases, the intake gates often are raised and lowered by means of an intake gantry, one at a time. Where the full complement of gates is provided, it is also common practice to provide individual fixed mechanical hoists for each gate, each hoist being controlled by the unit operator and the gate operation entirely independent of the crane. In the case of McNary, the extremely unbalanced hydraulic condition within the turbine water passages that would result from lowering the three gates for a unit, one at a time by means of the crane, under runaway conditions was considered to be excessive and to be avoided. Furthermore, inasmuch as the intake deck is not only a work area, particularly during flood conditions when large quantities of trash are encountered, but also a thoroughfare for project traffic across the dam, suitable locations for individual hoists could be found only at the expense of impeding traffic and loss of work area or excessive cost. Accordingly, each gate is provided with two vertical hydraulic cylinders pinned to the gate bottom and of a length approximately equal to the full height of the gate. The piston rods are attached to a support beam located above the gate and latched in the gate slot. The normal position of the gate when a machine is running is fully raised, latched to the support beam. Emergency closure of the gates is accomplished by the automatic system of pumps and valves which first raise the gates a sufficient amount to permit unlatching the gate from the support beam and then permits the gates to lower at a predetermined rate dependent upon the set rate of release of oil from the upper portion of the cylinders. Thus, the three gates for a unit are lowered simultaneously without undue disturbance due to unbalanced conditions in the turbine water passages and within a range of time of 3 to 7 minutes. The gates normally are raised to the latched, operating, position by pumping oil into the upper portions of the cylinders at an operating pressure of 2000 p.s.i. in about one hour. They may also be raised individually by the crane and transported to the gate repair pit for maintenance. There appeared to be no precedent for this type of hydraulic gate operating mechanism for operation on such a large scale and considerable difficulty was experienced both in fabrication of the cylinders and in ironing out unforeseen problems encountered during initial trial operation. A similar system is already under contract for the intake gates for The Dalles Powerhouse.

General Structural Design Considerations

Monolithic Construction

The powerhouse, in addition to its primary function for power production, must also act as a dam connecting the southernmost non-overflow section of the main concrete dam to the embankment on the Oregon shore. It is made up of 14 main unit bays, a station service bay, and an assembly bay, the total length of the building being 1422 feet. Each main unit bay and the station service bay is 86 feet long center to center of main piers and the total length of the assembly bay is 119 feet. The bay housing main unit No. 14 has a 13 foot superstructure extension beyond pier 15 to permit lowering of a crane hook on the north side of the generator housing. In the case of the main unit bays, the distance from center to center of main piers is governed by the requirement for scroll case space plus adequate pier thickness, while the lengths of the service and assembly bays were governed by the needs of the auxiliary equipment, erection space, and the desire to maintain uniform pier spacing. The lowest part of the draft tube is at elevation 180; the elevation of the intake deck is at elevation 361, a height of 181 feet. Structurally, each bay is separated from adjacent bays by contraction joints and must therefore

be independently stable. Furthermore, the intake structure, which in some plants is actually a gravity dam, is in plants of this type seriously lacking in mass due to the large area required for the turbine intake passages, the trash sluiceway and the intake gates. To obtain the maximum stability the intake is constructed monolithically with the powerhouse proper so that all loads are resisted by the bay as a whole. The entire structure is of reinforced concrete, with the exception of the structural steel roof trusses, purlins and bracing.

The contraction joints between monoliths are located on the centerlines of the main piers. They rise vertically from the base of the structure and are devoid of offsets of any kind. They serve to separate each bay from adjacent bays and to permit a certain amount of contraction due to shrinkage and cooling of the concrete without undue cracking. They are formed by treatment of the inner face of a half-pier before placement of the concrete for the adjacent half-pier. In line with the desire to eliminate cracking of the concrete about the water passages the peripheral walls of the concrete scrolls were divided into portions of lesser mass by means of keyed construction joints and a limitation placed on the rate of pouring as well as on the sequence of placement. Concrete for the roofs of the scroll cases was placed in a series of overlapped pie-shaped sectors, the maximum height of each of which was limited to 3-1/2 to 5 feet in 72 hours between lifts and with a limitation placed on the sequence of placement in adjacent sectors. In order that leakage be further eliminated or decreased to a minimum, rubber water stops were placed on vertical lift joints. Porous concrete drains are placed against the steel pit liner to carry leakage water off to the station drainage sump. From the experience at this plant to date, with several units in operation, it appears these extra provisions are very effective and are well worth the effort.

Loading Conditions

Separate stability analyses were required for the assembly bay, station service bay, main unit bays complete with turbines and generators, and incomplete or skeleton main unit bays which were to be in blocked-out form ready to receive embedded turbine parts as they were delivered. In order to obtain base pressures, for design purposes, under the intake slab and the much lower draft tube portions of the foundations of generator bays it was necessary to make stability analyses at these two levels. The assembly bay differs from the others in that four piers terminate at a point 37 feet upstream from the centerline of the units. This portion of the intake structure must therefore be stable without support from the rest of the bay. Live loads used in the design of the powerhouse are as follows:

a.	Roof slab	100 lb/sq. ft.
b.	Gallery floor slabs except at elev. 287	400 lb/sq. ft.
c.	Gallery floor slab at elev. 287	500 lb/sq. ft.
d.	Generator floor	1000 lb/sq. ft.
e.	Turbine floor	1000 lb/sq. ft.
f.	Control room roof and floor slabs	400 lb/sq. ft.
g.	Visitors gallery	200 lb/sq. ft.
h.	Wind (on vertical projection)	30 lb/sq. ft.

Some of the above loadings are acknowledged to be excessive but were adopted in order to provide resistance to vibration.

The main intake piers present many problems and are the most critical elements, from the standpoint of structural design, that occur in this type of powerhouse. Early construction at Bonneville indicated clearly that it is highly desirable to separate each bay by means of contraction joints extending

throughout the entire height of the structure in order to avoid unsightly crack-
ing and leakage. It follows that main piers are split into two halves by the
joint and this condition is directly responsible for one of the most difficult
problems in pier design. One of the particularly critical areas lies in the
water passage between the scroll case and the emergency gate slot. The pier
in this area is subject to two types of lateral load and a vertical load due to
the weight of the structure modified somewhat by the overturning effect of
hydrostatic pressure acting downstream.

Plant Arrangement

The arrangement of the McNary powerhouse is very similar to the Bonne-
ville plant in that the intake structure serves as an extension of the main dam
between a non-overflow section at midstream and the Oregon shore. Rein-
forced concrete is used to form the turbine water passages, for which the dis-
charge is approximately 17,700 cfs per unit at the critical head of 73.3 feet.
The selection of a fully inclosed type of construction and the arrangement,
size and locations of the assembly and station service bays were influenced
by the schedule for installation of units at the rate of one every three months.
The installation schedule also influenced the decision to provide a generator
room floor slab, heavily reinforced to withstand 1000 lb. per sq. ft. loading,
inasmuch as it was known that storage space would be at a premium for parts
and sub-assemblies of both turbines and generators and that should the gener-
ator floor have been eliminated the space on the turbine room floor would have
been entirely inadequate. A large open hatch has been provided in the floor of
the station service bay at elevation 287.0 (the main generator room floor level)
to facilitate the assembly of the Kaplan runners, shafts and inner head covers
at the turbine floor level (elevation 267.5). Generator rotor assembly is per-
formed at the 287.0 level in the assembly bay. An erection pedestal is pro-
vided for this purpose and space for another is available if found necessary.
The two 350 ton bridge cranes together are capable, by means of the use of
three lifting beams, of transporting the maximum lift comprised of a genera-
tor rotor and shaft to or from the assembly bay. All other lifts for either the
turbines or generators can be handled by either crane alone.

The superstructure provides adequate protection regardless of weather
conditions to workmen and equipment during the assembly and erection of non-
embedded turbine and generator parts. Its height was determined by the
necessity for transporting assembled runners and generator rotors from the
assembly bay to their respective permanent bays without interference with
intermediate units in operation. The superstructure is not exactly centered
over the longitudinal axis through the main units but is offset slightly down-
stream to facilitate this function. The location of the main units with respect
to the upstream face of the intake structure is dictated by the requirements of
the water passages. Both of these features, the offset of the superstructure
and location of the units, provide a space between the superstructure and the
intake structure in which galleries for the generator breakers and bus, control
cable trays, piping and pumps and heat exchangers for the transformers have
been located as shown in Figure 14. The main transformers are located on the
intake deck immediately over the galleries.

The upstream wall of the superstructure is comprised of a 2 foot concrete
curtain wall with pilasters below the bridge crane beam and rail. The curtain
wall between pilasters on the downstream side is 3 feet thick up to elevation
320, providing protection from maximum tailwater at elevation 303.0 (at
2,200,000 cfs) and from possible damage due to enemy military action. The
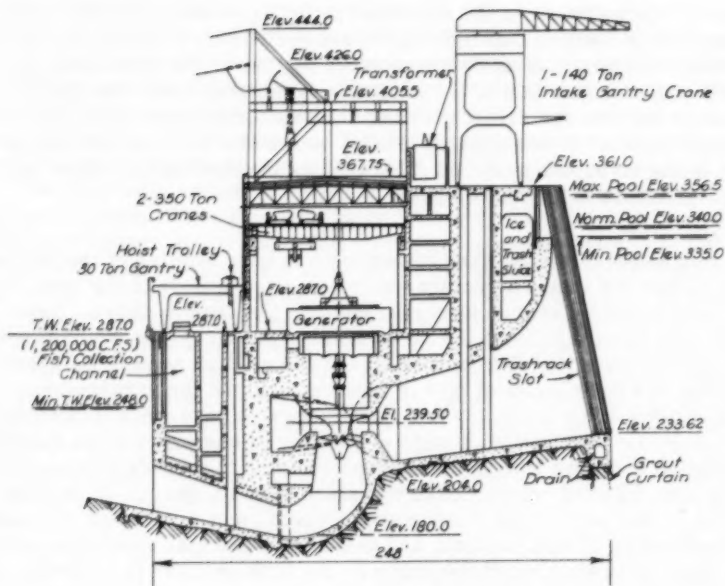reinforced roof slab is supported by steel trusses and purlins. Overlaying the

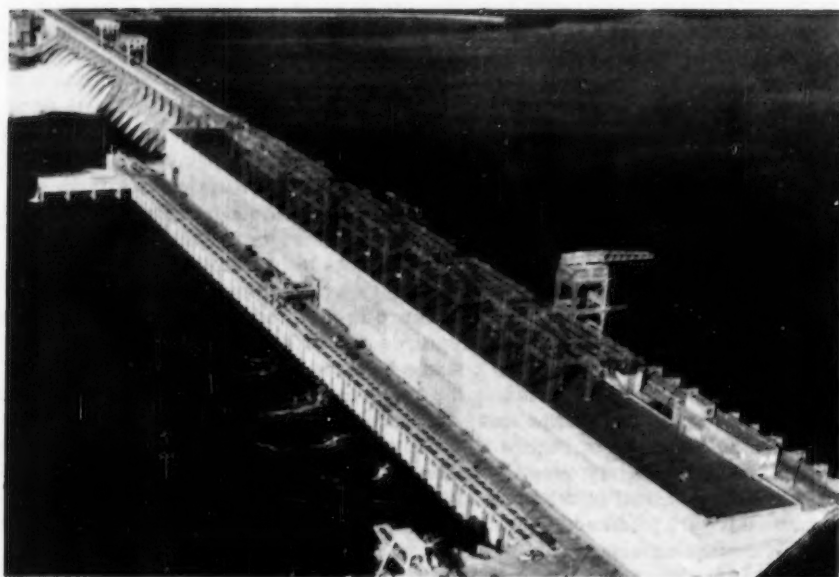FIG. 14 - SECTION THROUGH POWERHOUSE

FIG. 15. - POWERHOUSE AND SWITCHYARD

roof slabs are a 2 inch layer of fiberglass insulation, a water proof membrane and gravel.

Architectural Treatment

Exterior and interior architectural treatment of the McNary Powerhouse is extremely simple. Slight exterior rustication of the downstream wall of the superstructure was attempted in the form of shallow pilasters. The depth of this treatment, however, is such as to render the feature rather ineffective. Inasmuch as the intake deck and roof are practically at the same elevation, little could be accomplished architecturally on the upstream side. Concrete transformer cell blocks and the office building located on the intake deck appear as isolated blocks extending above the long low deck of the intake. Aside from the extensive use of paint on the walls and ceilings of all galleries and work areas, and provision of quarry tile on the entire generator floor, the architectural treatment inside the plant is practically limited to the facilities provided for the visiting general public. Since architectural treatment of a hydroelectric powerhouse is one of the least readily justifiable features, yet one on which a considerable expenditure of funds is required, the policy of the design office has leaned toward rather austere treatment in favor of features which will facilitate plant housekeeping and maintenance and avoid excessive expenditure of public funds. The public will, in general, be excluded from working areas but are invited to observe the plant from an observation platform above the visitors' lobby. From this location a general view is obtained of the full length of the generator room and a close-up view of the control room through a large window. In addition, in the lobby, there will be large photographic portrayals and sketches to illustrate various features of the plant. The complete absence of windows in the exterior walls of the plant is dictated by security requirements in line with the heavy wall. See Fig. 15. The power switching and transmission line pull-off structure on the roof is so located for utilitarian reasons and admittedly detracts from the architectural treatment of the plant. This choice was made after comprehensive studies of the economics of the various methods of transmitting power away from the plant, showing an estimated saving in excess of one million dollars over any acceptable alternative.

The technical papers published in the past year are presented below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways (WW) divisions. For titles and order coupons, refer to the appropriate issue of "Civil Engineering" or write for a cumulative price list.

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SEPTEMBER: 484(ST), 485(ST), 486(ST), 487(CP)[c], 488(ST)[c], 489(HY), 490(HY), 491(HY)[c], 492(SA), 493(SA), 494(SA), 495(SA), 496(SA), 497(SA), 498(SA), 499(HW), 500(HW), 501(HW)[c], 502(WW), 503(WW), 504(WW)[c], 505(CO), 506(CO)[c], 507(CP), 508(CP), 509(CP), 510(CP), 511(CP).

OCTOBER: 512(SM), 513(SM), 514(SM), 515(SM), 516(SM), 517(PO), 518(SM)[c], 519(IR), 520(IR), 521(IR), 522(IR)[c], 523(AT)[c], 524(SU), 525(SU)[c], 526(EM), 527(EM), 528(EM), 529(EM), 530(EM)[c], 531(EM), 532(EM)[c], 533(PO).

NOVEMBER: 534(HY), 535(HY), 536(HY), 537(HY), 538(HY)[c], 539(ST), 540(ST), 541(ST), 542(ST), 543(ST), 544(ST), 545(SA), 546(SA), 547(SA), 548(SM), 549(SM), 550(SM), 551(SM), 552(SA), 553(SM)[c], 554(SA), 555(SA), 556(SA), 557(SA).

DECEMBER: 558(ST), 559(ST), 560(ST), 561(ST), 562(ST), 563(ST)[c], 564(HY), 565(HY), 566(HY), 567(HY), 568(HY)[c], 569(SM), 570(SM), 571(SM), 572(SM)[c], 573(SM)[c], 574(SU), 575(SU), 576(SU), 577(SU), 578(HY), 579(ST), 580(SU), 581(SU), 582(Index).

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JANUARY: 583(ST), 584(ST), 585(ST), 586(ST), 587(ST), 588(ST), 589(ST)[c], 590(SA), 591(SA), 592(SA), 593(SA), 594(SA), 595(SA)[c], 596(HW), 597(HW), 598(HW)[c], 599(CP), 600(CP), 601(CP), 602(CP), 603(CP), 604(EM), 605(EM), 606(EM)[c], 607(EM).

FEBRUARY: 608(WW), 609(WW), 610(WW), 611(WW), 612(WW), 613(WW), 614(WW), 615(WW), 616(WW), 617(IR), 618(IR), 619(IR), 620(IR), 621(IR)[c], 622(IR), 623(IR), 624(HY)[c], 625(HY), 626(HY), 627(HY), 628(HY), 629(HY), 630(HY), 631(HY), 632(CO), 633(CO).

MARCH: 634(PO), 635(PO), 636(PO), 637(PO), 638(PO), 639(PO), 640(PO), 641(PO)[c], 642(SA), 643(SA), 644(SA), 645(SA), 646(SA), 647(SA)[c], 648(ST), 649(ST), 650(ST), 651(ST), 652(ST), 653(ST), 654(ST)[c], 655(SA), 656(SM)[c], 657(SM)[c], 658(SM)[c].

APRIL: 659(ST), 660(ST), 661(ST)[c], 662(ST), 663(ST), 664(ST)[c], 665(HY)[c], 666(HY), 667(HY), 668(HY), 669(HY), 670(EM), 671(EM), 672(EM), 673(EM), 674(EM), 675(EM), 676(EM), 677(EM), 678(HY).

MAY: 679(ST), 680(ST), 681(ST), 682(ST)[c], 683(ST), 684(ST), 685(SA), 686(SA), 687(SA), 688(SA), 689(SA)[c], 690(EM), 691(EM), 692(EM), 693(EM), 694(EM), 695(EM), 696(PO), 697(PO), 698(SA), 699(PO)[c], 700(PO), 701(ST)[c].

JUNE: 702(HW), 703(HW), 704(HW)[c], 705(IR), 706(IR), 707(IR), 708(IR), 709(HY)[c], 710(CP), 711(CP), 712(CP), 713(CP)[c], 714(HY), 715(HY), 716(HY), 717(HY), 718(SM)[c], 719(HY)[c], 720(AT), 721(AT), 722(SU), 723(WW), 724(WW), 725(WW), 726(WW)[c], 727(WW), 728(IR), 729(IR), 730(SU)[c], 731(SU).

JULY: 732(ST), 733(ST), 734(ST), 735(ST), 736(ST), 737(PO), 738(PO), 739(PO), 740(PO), 741(PO), 742(PO), 743(HY), 744(HY), 745(HY), 746(HY), 747(HY), 748(HY)[c], 749(SA), 750(SA), 751(SA), 752(SA)[c], 753(SM), 754(SM), 755(SM), 756(SM), 757(SM), 758(CO)[c], 759(SM)[c], 760(WW)[c].

AUGUST: 761(BD), 762(ST), 763(ST), 764(ST), 765(ST)[c], 766(CP), 767(CP), 768(CP), 769(CP), 770(CP), 771(EM), 772(EM), 773(SA), 774(EM), 775(EM), 776(EM)[c], 777(AT), 778(AT), 779(SA), 780(SA), 781(SA), 782(SA)[c], 783(HW), 784(HW), 785(CP), 786(ST).

SEPTEMBER: 787(PO), 788(IR), 789(HY), 790(HY), 791(HY), 792(HY), 793(HY), 794(HY)[c], 795(EM), 796(EM), 797(EM), 798(EM), 799(EM)[c], 800(WW), 801(WW), 802(WW), 803(WW), 804(WW), 805(WW), 806(HY), 807(PO)[c], 808(IR)[c].

c. Discussion of several papers, grouped by Divisions.